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The Opposition of Some Automobile Clubs to "Gas Taxes"

When Illinois recently passed a gasoline tax law all but two of our states had adopted this means of financing at least part of their road improvement. But Illinois is again without this source of income, for the state supreme court has declared the law invalid. Certain automobile clubs fought the law in the state legislature and afterward in the courts, acting on the theory that automobile owners can somehow have their cake without paying for it. It may be true that in Illinois a disproportionate part of gasoline tax was spent for roads "down state," but that was a condition that could have been remedied. Judging by billboard and other propaganda the real cause of the fight against the gasoline tax was the desire of certain automobile club officials to give evidence that they earn their salaries.

Since the law was unconstitutional, it is well that it was opposed by some one, but the opposition that it met in the legislature was from the same source; and it does little credit to the tax dodgers who were behind it. Judging from the number of states that have passed gasoline tax laws, the popular verdict is overwhelmingly favorable to this means of financing road improvement. Like every form of tax, it is not perfectly equitable; but it has fewer defects in that respect than any substitute that has yet been suggested. It has the outstanding merit of making the tax on each person roughly proportioned to his use of the highways, and at the same time of placing so light a burden upon him at any time as not to render the tax obnoxious. Those of us who struggled for years in the nineties and later to secure even small appropriations for road work, realize the great difficulty of persuading tax payers to vote adequate sums for road improvement. Had the gasoline tax been devised earlier, we should now have a much greater mileage of satisfactory roads, and much less congestion because of inadequate widths.

We venture to believe that the way officers of the automobile clubs who fought the Illinois "gas tax" are loud enough in their complaint about the Saturday and Sunday congestion of roads near Chicago. Do they expect the abutting property owners to vote millions of taxes upon themselves to relieve that congestion? If not, where is the much needed money going to come from?

Why Real Wages Have Increased 35 Per Cent in 14 Years

Eight years ago the editor published in "Engineering and Contracting" two articles that contained a study of commodity prices and wages in America for a century, and in Great Britain for six centuries. It was shown that per capita productivity both in America and England had steadily increased so that at the end of a century it was nearly five fold what it had been. Curiously enough the per capita productivity prior to about the war of the revolution had been almost stationary in Britain for five centuries, indicating that not until "the power age" was ushered in by the steam engine did a marked and steady advance in productivity begin.

Rarely has there been a decade in which per capita productivity has not increased fully 20 per cent, so it is not startling to read that the buying power of the average factory worker's weekly "real wage" has increased 35 per cent during the last 14 years. Not a few writers on economics have spoken of such an increase as being without precedent, but that is an error.

According to the National Industrial Conference Board the cost of living has increased 61 per cent since July, 1914, and the average weekly wage of factory workers has increased 118 per cent, the hourly wage having increased 131 per cent. Hence the buying power of the weekly wage has increased to 218÷161=

1.35, or an increase of 35 per cent in 14 years, or 2.5 per cent per annum.

It is probable that the buying power of all persons in gainful occupation, including farmers, clerks, etc., has increased somewhat less than 2.5 per cent per annum, possibly not more than 2 per cent annually. But even so, the gain in "real wages" has been very satisfactory.

The real wage is measured by dividing the "index" of wages by the "index" of the cost of living; and it is the relative buying power of wages. The articles above mentioned (which are to be found in full in Gillette's Handbook of Construction Costs) show conclusively that "real wages" have gone hand in hand with per capita productivity. Only because such productivity has increased have "real wages" increased. Economists had long believed this to be true, but there had been no adequate statistical proof of its truth until the studies above mentioned were made.

Those studies also resulted in the discovery of two economic laws. First, that the trend of money wages is closely parallel with the trend of per capita money. Second, that that the trend of commodity prices follows the trend of wages divided by per capita productivity. There is another factor that causes moderate departures at times from these trends, namely the "velocity of circulation of money," or what might be called the business cycle factor. It is measurable by dividing total bank clearings outside of New York City by average bank deposits. New York City clearings must be eliminated in order not to introduce the effect of stock exchange transactions.

Since it is now demonstrable that average real wages are entirely dependent upon the per capita output of our people, it follows conclusively that most of the efforts of labor unions have been futile. Here and there a strong union has occasionally forced the wages of its members above the general level, and it has seemed to them that they were winning a battle against the employers. They failed to realize that unless the general level of wages could be raised, their advantageous position could not be long maintained. The bituminous miners of Ohio, Illinois and Indiana managed to force wages above the general level, and most of them have lost employment by doing so.

Probably their leaders would argue that they have failed in their effort only because there was not complete unionization of all the workers engaged in producing fuel. But even had that been the case, it would have eventually become necessary to unionize every other industry if all wages were to be increased by coercion; and then it would be found that coercion of employers is fatal to an increase in productivity, without which real wages can not increase. Great Britain is today a good example of how union coercion works. Political nostrums are being tried there, but with no better result. There has yet to be found any scheme so effective in raising real wages as the untrammeled competition of keen business men. Successful employers necessarily bid against one another not only for raw materials but for employees; and as their operations expand this bidding increases, automatically increasing wages.

A study of the U. S. Census of Manufacturing shows that for half a century the net earnings have varied from 10 per cent of the gross sales. Unless one adds percentages to cover items omitted by the Census, such as depreciation, taxes, rents, etc., it seems as if much more than 10 per cent had been averaged; and perhaps this failure to interpret correctly the published statistics is largely responsible for the delusion of labor union leaders who so frequently talk of the enormous profits of manufacturers. Even the most successful manufacturer that the world has ever seen, Henry Ford, has averaged only 10 per cent profit on his sales, but he has conducted his business on such an enormous scale that he has outstripped in wealth every other Croesus.

Since nearly all of the 90 per cent that Ford pays for raw material, transportation, taxes, etc., ultimately finds its way into pay envelopes, it follows that if he were deprived of the 10 per cent profit, only 10 per cent would be added to the average pay envelope. At the rate of progress during the past 14 years, 10 per cent has been added to the real wage of manufacturing employes every 4 years. Hence all that our socialistic and unionistic friends have been clamoring for, namely a distribution of employer's profits, has virtually been accomplished every 4 years. It is true that as fast as the old 10 per cent profit is distributed in wages a new 10 per cent is created, but what sane worker can object to an economic system that makes him the constant gainer of all but a moiety of the fruits of industrial progress?

If the average worker will commune with his conscience he will admit that neither by increase in his hours of work, nor by increase in his exertion while working, nor his own inventiveness, or other brain activity, has his real wage been raised 35 per cent in 14 years.

If he will study statistics he will see that this 35 per cent is more than three-fold the entire profit of the average employer 14 years ago. Where, then, has the gain come from? To whom is he indebted for this windfall? There is but one possible reply: Every American employe is indebted for his increased real wages to the use of brain power by our scientists, our engineers and our business executives. Upon scientists rests the burden of discovering the laws of nature that have made possible the electric generator, the telephone, the radio, the motor car, the aeroplane. Upon our engineers rests the burden of so applying the laws of nature that economic machines and structures, can be made. Upon our business executives rests the burden of financing the enterprises, directing the industrial armies, and selling their products to the public at such a profit that further financing for purpoess of further industrial expansion may be possible.

To that great triumvirate of scientists, engineers and executives, our American productivity is attributable; and because of that unexampled productivity, American wages are the envy of the world.

H. P. Gillette

The Design of Small Sewage Treatment Works at Gaithersburg, Md.

Separate Sludge Digestion and Modified Form of Imhoff Tank

By HARRY R. HALL

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THIS paper describes the design of a small sewage treatment works serving the communities of Gaithersburg and Washington Grove, two small detached areas under the jurisdiction of the Washington Suburban Sanitary Commission. The plant has a nominal capacity of 250,000 gal. per 24 hours for a population of 2,500. It is of the usual sprinkling filter type, with sedimentation preceding and following the sprinkling process. Outstanding features are the application of separate sludge digestion and the adoption of a modified form of Imhoff tank for primary and final sedimentation, which resulted in a material saving in cost over the use of two-story deep tanks for combining the processes of sedimentation and sludge digestion.

Introduction.—Gaithersburg and Washington Grove, with a combined population of approximately 2,000, were made a part of the Washington Suburban Sanitary District by legislative enactment in 1924, to permit them to secure water and sewer systems under the administration of the District. They are separate communities about a mile apart and nearly 8 miles from the original Sanitary District. Gaithersburg is a residential town of 1,300 people situated on the Old National Pike. Washington Grove is essentially a summer resort, originally established as a camp meeting ground. Its summer population is about 700. The permanent residents amount to considerably less than one-half this number.

These communities are too far removed from the main part of the sanitary district to be served by the sewer system of the district and neither is situated near a stream sufficiently large to receive sewage without treatment of some kind. Consequently, a disposal plant necessarily became part of the sewage project.

The topography of Gaithersburg is such that about two-thirds of the area drains to the treatment works by gravity. The sewage from the remainder, together with that from Washington Grove, is pumped at four small pumping stations to the gravity area. In this way all sewage from the combined communities is concentrated at one point for treatment.

The total length of sewers, not including force mains and the outfall to the treatment works, is 7.2 miles. The outfall, including 430 ft. of 8 in. inverted siphon, is of 12 in. and 15 in. pipe 1.2 miles long. It has a capacity of 2,300,000 gal. per 24 hours, which is sufficient to care for the estimated ultimate requirements of the Gaithersburg area, or approximately 7,000 people. The Washington Grove area will be disconnected from the present outfall at some time in the distant future and drain to the works through another valley when conditions require a sewer there.

Location of Treatment Work.—The site of the treatment works is about one mile from the corporate limits of Gaithersburg in the valley of Muddy Branch, a small stream with a watershed of 2.5 square miles, originating just south of Washington Grove and discharging into Great Seneca Creek, a tributary of the Potomac River, 8 miles distant from the disposal plant. It is over ¼ mile from the nearest road and the nearest house. The entire countryside is devoted to farming and is sparsely settled. The location is well adapted for sewage treatment as the plant is hidden from view by protecting hills, except on the south or stream side. The area of the site is large enough to provide for the development of more than three times the capacity of the present works.

General Description.—In the selection of the type of plant it was attempted to reconcile strict economy in cost with adequacy of treatment. These two small communities were undertaking a considerable tax burden in the recent installation of water and sewer sys-

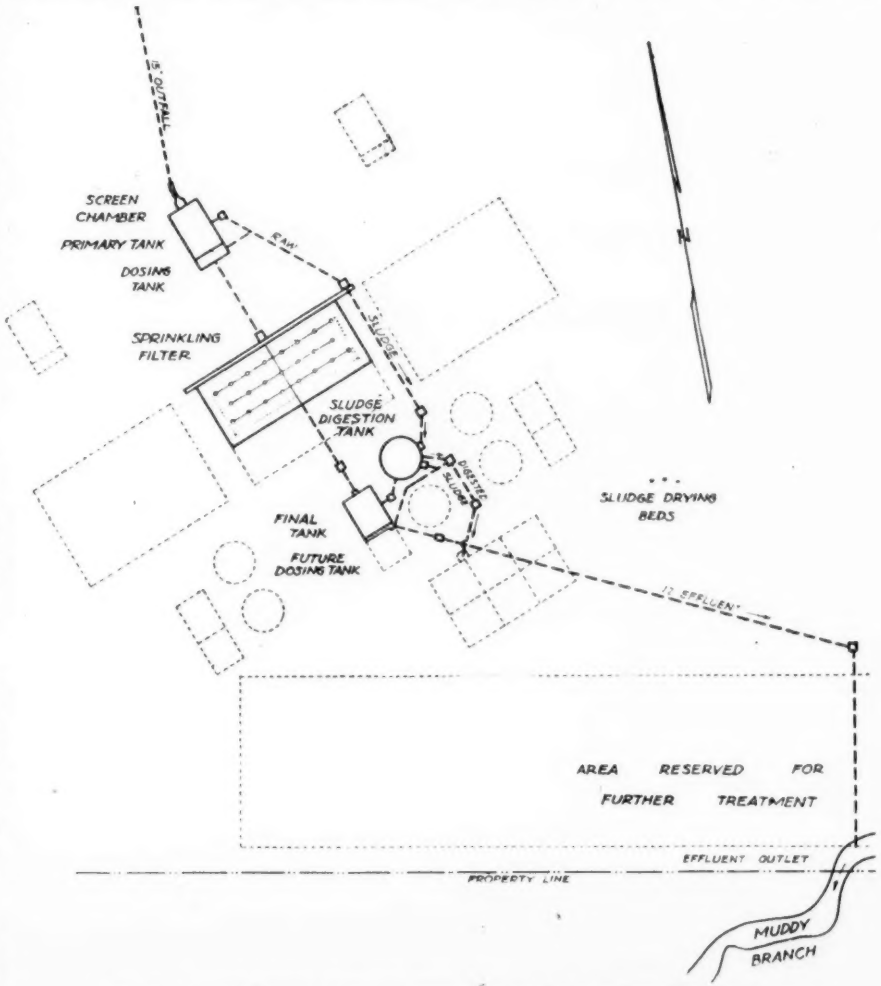


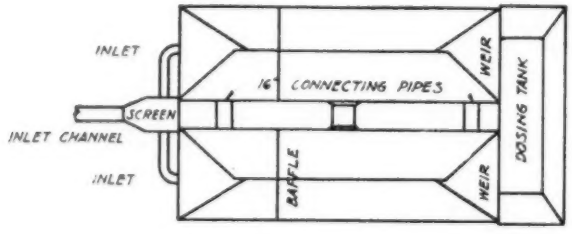
Fig. 1.—General Layout of Sewage Treatment Works at Gaithersburg, Md.

tems. A too costly type of sewage treatment works might make that burden excessive. On the other hand as the country all about is occupied by excellent farms, most of which carry on a dairy business, and as practically all the streams flow through dairy farms somewhere along their courses, a reasonably complete method of treatment was necessary. The plant was to be a small one. It would be operated by the same agency that designed it and proper attention to operation could be assured. No electric current was available anywhere near the site, so that mechanically operated devices, such as would be required in the activated sludge process or in those methods of sedimentation involving the mechanical removal of sludge, could not be used without installation of another kind of power. It was felt advisable to keep this small plant free from machinery so as to have nothing of that character to require attention.

A sprinkling filter installation with primary and final sedimentation, so laid out that further treatment could be applied later, if necessary, when the flow of sewage became great enough, seemed best to fit these conditions.

The plant, as designed and now nearing completion in construction, is composed of a small screen chamber at the inlet of the primary sedimentation tank, primary tank and dosing chamber, sprinkling filter, final sedimentation tank, sludge digestion tank and sludge drying beds. Only one-half the sprinkling filter and one sludge tank for one-half the capacity of the plant are being built at this time. The sludge beds for one-half the plant capacity will be built this summer. These will be added to or duplicated when needed. The effluent from the final tank will be discharged directly into Muddy Branch.

Figure 1 shows the general arrangement of the plant and the manner in which extensions may be made. It should be noted that the design of the units is balanced. Any expansion, except for completing those units of the present works not built full size at this time, would involve completely new units of each kind and not an increase in size of one or the other. It is believed that this feature allows the greatest amount of latitude in the future design and will permit the fullest advantage to be taken of later improvements in methods of sewage disposal.



PLAN AND SECTIONS OF ADOPTED DESIGN OF PRIMARY SEDIMENTATION TANK

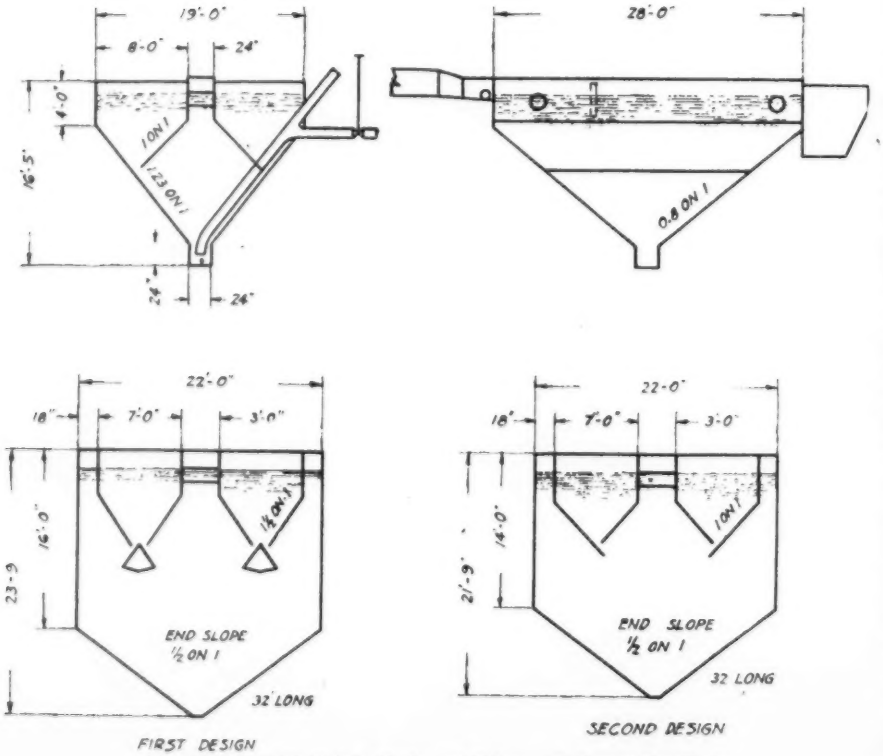


Fig. 1.—Types Studied in Design of Primary Sedimentation Tank

Discussion of Design of Sedimentation Tanks.—Figures 2 and 3 show a comparison of types of primary and final sedimentation tanks considered with the designs finally adopted. The first design studied provided for Imhoff tanks with sedimentation periods of 1½ and 1 hours and sludge capacities of 2 and 1 cu. ft. per capita respectively, with slopes on the inner walls separating the sedimentation and sludge digestion compartments of 1½ on 1 and a minimum bottom slope of ½ on 1. During the discussion of Mr. Schaetzle's paper on sewage disposal at the meeting of the Maryland Water and Sewage Association last year the question of using flatter inner-wall slopes was given much attention. Following this a revised design was prepared which changed the inner-wall slopes to 1 on 1. Keeping the factors of sedimentation and sludge the same as in the first case it was found that the total depths of the primary and final tanks could be decreased by 2 ft. and 1 ft. 9 in., respectively, with a total saving in cost of nearly $2,000.

A third type involving separate sludge digestion was then considered. The 2-story type of tank and the same period of sedimentation were kept, but the sludge compartment was reduced in size to retain sludge over a short period only until it could be removed to the digestion tank. This further reduced the depths of both tanks by 5 ft. 4 in. and 3 ft. 4 in. from the second type studied, or a total reduction from the first case of 7 ft. 4 in. and 5 ft. 2 in.

Comparisons of costs showed that the modified form of Imhoff tank, together with the necessary sludge digestion tanks with the same sludge capacity as the combined capacities of the primary and final Imhoff tanks previously studied, could be constructed at a saving in cost of approximately $3,000 over the first tank design considered and $1,000 over the second. This saving on such a small installation was too great to be ignored, particularly as even a greater saving in first cost could be made by building only one sludge tank now, and consequently the primary and final sedimentation tanks shown, and later more fully described, were adopted.

Screen Chamber, Primary Sedimentation and Dosing Tanks.—These were built as one structure. Before entering the primary tank the sewage flows through a small screen chamber 5 ft. long, 3 ft. wide and 2 ft. deep with a sloping bottom, containing a bar screen composed of ½ in. by 1⅛ in. bars with a clear opening of 1½ in. set at an angle of 35° with the horizontal.

The primary tank has a theoretical detention period of 1½ hours, based on the average sewage flow. The sludge

reception compartment has a capacity of 309 cu. ft. below the slots. In inside dimensions the tank is 28 ft. long, 19 ft. wide and approximately 16 ft. 5 in. deep to the bottom of a 2-ft. square sump. The side and end walls are vertical for a distance of 4 ft. from the top, from which point they converge toward the bottom, the side walls at a slope of 1.23 on 1 and the end walls at 0.8 on 1, all meeting at the top of the sump 2 ft. above the bottom of the tank. Figure 4 shows a view of the partially completed tank.

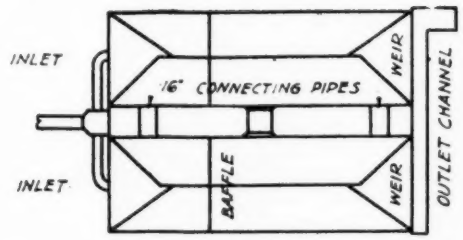
There are two sedimentation chambers 8 ft. wide divided by a central compartment corresponding to the gas vent in the ordinary type of Imhoff tank. The two chambers are connected near each end by 16-in. pipes to minimize short circuiting through the slots in case the level in one becomes higher than in the other.

The central compartment is 2 ft. wide and extends the full length of the tank. Its walls rise 6 in. above the main walls of the tank, and extend vertically 4 ft. down from the top where they join the 1-on-1 sloping inner walls to complete the separation of the sedimentation compartment above from the sludge compartment beneath. The inner walls extend to within 6 in. of the outer walls, providing slots of that width. With the long sloping outside walls it will be possible to scrape them all the way to the bottom of the tank, thus providing agitation for the sudge in addition to cleaning the walls.

Sewage enters the tank at the center of the inlet end of each sedimentation chamber, passes under a baffle and flows from the tank directly into the dosing tank over a full-width weir at the outlet end of each chamber. Sludge is removed hydraulically through an 8-in. pipe projecting into the sump at the bottom of the tank, with its outlet 4 ft. below the sewage surface, and is conveyed through a pipe directly to the digestion tank. To assist the flow of sludge settled sewage can be delivered directly to the sludge line by opening a gate in the dosing tank which controls a connection to the sludge line.

The dosing tank has a capacity of 3,400 gal. It is provided with a single 10-in. Miller trapless siphon for automatic intermittent application of the settled sewage to the sprinkling filter under a maximum head of 6.6 ft. above the sprinkler nozzles.

Final Sedimentation Tank.—This unit is similar in design to the primary tank. It has a theoretical detention period of 1 hour, based on the average sewage flow. The sludge compartment has a capacity of 180 cu. ft. The tank is 23 ft. 6 in. long and 17 ft. wide inside. The depth to the bottom of the 2 ft. square sump is approximately 14 ft. 7 in. The vertical walls are 4 ft. deep, the lower sidewalls have a slope of 1.15 on 1 and the endwalls 0.8 on 1. A central compartment 2 ft. wide separates the two 7 ft. wide sedimentation



PLAN AND SECTIONS OF ADOPTED DESIGN OF FINAL SEDIMENTATION TANK

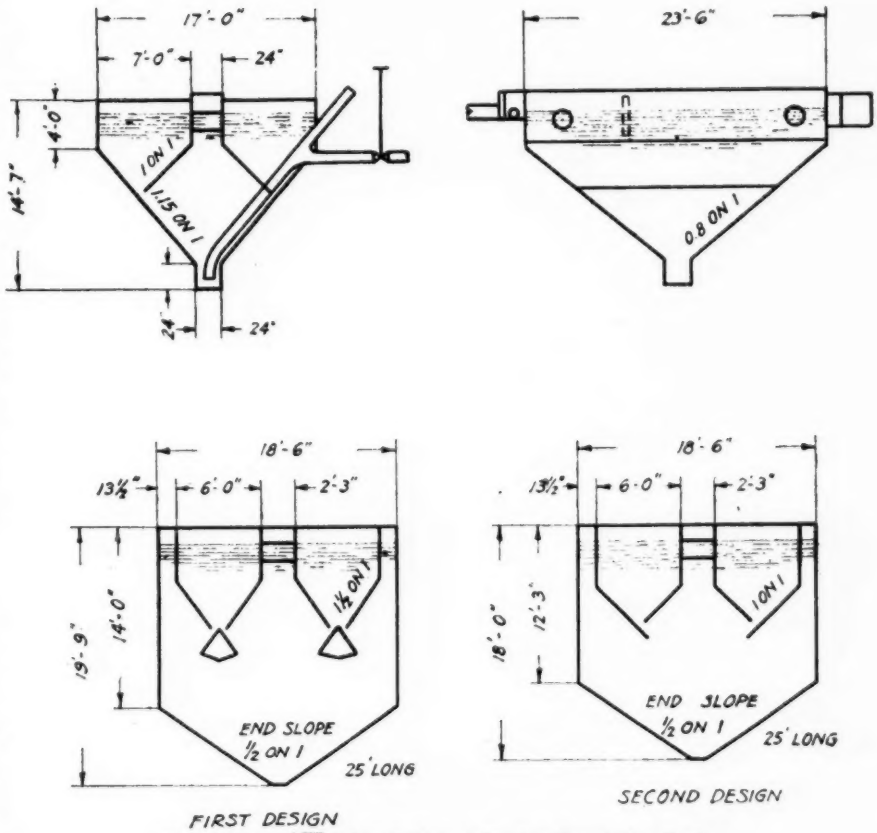


Fig. 3.—Types Studied in Design of Final Sedimentation Tank

Fig. 4.—View of Primary Tank

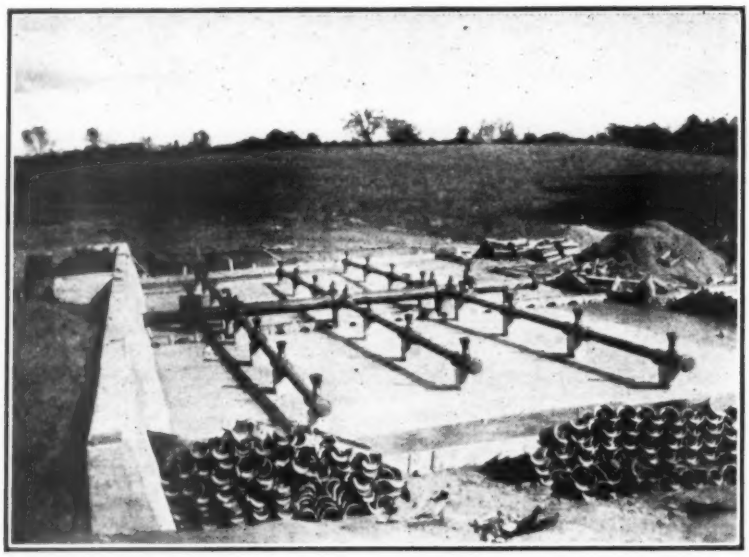


Fig. 5.—View of Sprinkling Filters Before Lateral Underdrains Were Placed

chambers and the inner sloping partition walls on 1 on 1 slope complete the separation of the detention and sludge compartments.

The effluent from the final tank is discharged directly into Muddy Branch without further treatment. A connection is arranged in the effluent channel to provide for diverting part of the flow to the sludge line for flushing, if necessary.

Sprinkling Filter.—This unit, as built, is approximately 42 ft. 3 in. long by 90 ft. 6 in. wide. The following description is based on the full-size unit. When increased to full size it will be 69 ft. 6 in. by 90 ft. 6 in. in plan with an effective stone area of 4,116 sq. ft., 56 ft. long by 73 ft. 6 in. wide. The actual surface of the stone extends 3 ft. beyond these limits on each side. The rate of application based on the average sewage flow is 2.65 Mg. per acre daily. The filter medium is limestone, quarried at Martinsburg, W. Va., varying in size from 1 in. to 2½ in. It has an average depth of 5.6 ft. and is supported on a 4-in. concrete slab underlying the entire bed and sloping to the main underdrain which runs lengthwise through the center. The floor for a distance of 6 ft. in from the edge was covered with two coats of Inertol. Figures 5 and 6 show views of the partially completed installation.

The stone is sloped at 45° on three sides of the filter instead of being confined by a wall. The fourth or upper side, where the excation is deepest, required a wall to retain the earth. The omission of the walls on three sides increased the amount of stone filtering material and concrete bottom slab, but resulted in a net saving of $400 on the half unit.

Distribution of the applied sewage is effected by means of a main 12-in. cast iron distributing pipe, running lengthwise through the center of the bed, from which lead 12 lateral 6 in. lines feeding through 3 in. risers 38 full-spray circular and 8 half-spray or semi-circular ⅞ in. diameter nozzles spaced on 10 ft. 6 in. centers. The nozzles are staggered on adjoining lines. This distributing piping is laid about 3 ft. 6 in. below the surface of the stone, supported on concrete piers, in order to provide some protection against freezing of the lateral lines during the severely cold winter weather in the early days of the plant when the flow is small and the discharge periods of the siphon are far apart. The extra expenditure for this protection was deemed worth while.

The underdrainage system consists of 6 in. vitrified channel pipe in 2 ft. lengths laid in an inverted position with 1 in. open joints on the 4 in. concrete bottom at approximately 2 ft. centers. These extend from the outer edge of the stone to the main underdrain with their upper ends open for inspection and cleaning. The main underdrain is composed of 12 in. by 6 in. vitrified double T-branches in 2-ft. lengths with the lower half encased in concrete. There are manholes at the upper and lower ends of the main drain to provide for inspection and cleaning.

The main underdrain was laid for its full length, but the remaining parts were built only for half the capacity. When the sewage flow increases the floor will be enlarged and the distribution and under drainage systems extended without disturbance of the operation of the plant in any way. During the early period of the works the dischanging rate of the siphon will be but one-half that for which it was designed.

Sludge Digestion Tank.—This tank has been designed for 1,250 persons at 3 cu. ft. of sludge per person. On this basis two tanks of the size of this will be required. It is cylindrical in shape with a conical bottom, the slope of which is 0.57 on 1. The cylinder 22 ft. in diameter and 11 ft. 9 in. deep inside. The total inside depth of the tank is 17 ft. 9 in. Sludge from the two sedimentation tanks is discharged into the digestion tank near the center and about 6 ft. below normal water surface. Provision is made for removing supernatant liquid through valved outlets 0.20, 2.00 and 3.90 ft. below normal water level. This liquid will be discharged onto one of the sludge drying beds which will have a greater depth of sand for the purpose than the others.

Digested sludge is removed hydraulically through an 8-in. pipe rising from the low point in the bottom. Agitation of the sludge will be effected manually by the use of a scraper working on the sloping bottom.

Sludge Drying Beds.—These beds have not been constructed, but will be built this summer. Three beds will be built at first, each 20 ft. by 20 ft. in plan, designed for 1 cu. ft. of digested sludge per person. They will have 12

Fig. 6.—View of Sprinkling Filter Showing Placing of Lateral Underdrains and Stone

in. of graded gravel and 6 in. of sand overlying underdrains of 4 in. open-joint unglazed farm tile laid 6 ft. 8 in. apart and discharging into the main effluent pipe which passes through the sludge bed area. One bed will have a 12-in. depth of sand to receive the liquid removed from the sludge digestion tank.

Costs.—Construction work on the plant was suspended at the approach of severe winter weather, with most of the work completed, except the sludge drying beds which were not in this construction program. There still remain some small tasks to be finished, but the work is sufficiently along so that a fairly close approximation of the final cost can be reached. With the exception of the sludge beds the cost of the works as planned for first construction will be about $23,500. The estimated cost to build three sludge drying beds, which will correspond with the present installation, is $2,200.

Using the bid unit prices on this contract it is estimated that completion of the sprinkling filter and the construction of a duplicate sludge digestion tank to bring these units of the plant to full capacity will cost $9,600. The addition of three more sludge drying beds is estimated to cost $2,300 more.

The estimated cost of the completed plant with all units brought to full size is approximately $37,600. This figure is subject to change, of course, depending upon material prices and labor conditions at the time the remaining work is done. All the above cost figures include engineering and administration, but no land or right of way.

Personnel.—The design and construction supervision of this plant were carried out by the engineering force of the Washington Suburban Sanitary District, of which Robert B. Morse is chief engineer. His suggestions and guidance were responsible for the prosecution of the studies which produced the saving in cost. The studies and designs were made under the immediate direction of the writer, who also exercised general supervision over construction. The construction department of the district, of which H. Roland Devilbiss is department engineer, immediately supervised the construction work. Noble L. Owings was engineer-in-charge on the work. Ligon and Ligon, of Baltimore, were the contractors.

Acknowledgment.—The foregoing is a paper presented April 11 before the Second Annual Convention of the Maryland Water and Sewerage Association.

Joint Outlet Sewer for New Jersey Cities.—Plans and specifications for an outlet sewer for seven New Jersey municipalities have been approved by the cities interested. The sewer which will relieve the Second River of pollution, will empty into the Passaic Valley trunk sewer.

Water Softening and Purification at Columbus, O., in 1927

In the 1927 report of the Division of Water of Columbus, O., Charles P. Hoover, chemist in charge of the water softening and purification works summarizes the water treatment work for that year as follows:

During the year an average of 28.5 million gallons of water was softened and filtered each day. This represents 98 gal. per capita per day.

Hardness of Water.—The hardness of the water was reduced from an average of 230 parts per million to an average of 87 parts per million. A reduction of 62 per cent.

Quantity of Chemicals Used and Their Cost.—To reduce the hardness of the water from an average of 230 parts per million to 87 parts per million, 2,514 tons of soda-ash and 7,215 tons of lime were required. The price paid for soda-ash was $28.60 per ton, and the price paid for the lime was $6.69 per ton, making the total cost for soda-ash $71,900.40, and the total cost for lime $48,268.35. The cost of the lime, soda-ash, alum and chlorine per one million gallons of water treated amount to $14.00.

Turbidity and Color Removed.—The average turbidity or muddiness of the river water during the year was 121 parts per million, whereas the turbidity of the filtered water was zero. The average turbidity was higher during the past year than in former years and this is due to the O'Shaughnessy dam which holds back the flood water, causing the water coming to the plant to be muddier for longer periods of time. The average color present in the above water, derived from decay of vegetation, was 21 parts per million. By the combined use of lime, soda-ash and alum, the color was reduced in the filtered water to 1 part per million. A color of one part per million is not noticeable to the consumer.

Typhoid Fever.—There were six deaths from typhoid fever during the year 1927, three of which were resident and three non-resident. Figuring the population at 291,400 (state bureau of vital statistics estimate) would give the city a total death rate from typhoid fever of 2.0 per 100,000, a resident death rate of 1.0 per 100,000 and a non-resident death rate of 1.0 per 100,000.

Carbonation Plant.—The carbonation plant was built and put in operation in August 1927.

The gas making plant consists of six principal parts: a gas producer, a gas burner, a return tubular boiler, a steam driven air compressor, a combination scrubber and dryer and a diffuser.

In operation, coke is burned in the gas producer, which is a closed furnace with controlled air supply furnished by a blower. The products of combustion pass from the gas producer to a gas burner where they are mixed with air and burned to complete combustion under a boiler. The steam produced in the boiler is used to drive the air compressor and blower so as to make a closed cycle for the process. The air compressor takes its suction from the boiler stack through a scrubber and dryer and forces the washed gases through diffuses into the water to be carbonated.

Table I shows the average number of pounds of CO_2 applied per million gallons of water treated, the number of pounds of coke burned per million gallons of water treated, also the average alkalinities of the settled water before carbonation, after carbonation and after filtration, for the months of September, October, November and December, 1927.

The results show that the phenol alkalinity has been reduced by carbonation from an average of 33 parts per million to 7 parts per million or that the caustic alkalinity has been reduced from 6 parts per million to 0 and the normal carbonates from 54 parts per million to 14 parts per million.

Practical results of carbonation at the Columbus plant will be discussed after more data has been obtained. The plant has operated successfully and the only trouble that has been encountered has been due to the corrosive action of the gases on the valve of the air compressor used to force the washed gases through the diffusers.

Hydrants Broken by Automobiles.—The 62nd annual report of the Water Board of Cambridge, Mass., for the year ending March 31, 1927, contains the following:

"It has been necessary to repair 30 hydrants this year; 25 of these were broken by automobiles. We have been enabled, through the cooperation of the Police Department, to trace the owners of 80 per cent of these cars, and the cost of the repairs of the hydrants has been paid by them or their representatives."

Table I.—Results of Operation of Carbonation Plant at Columbus, O.

Date 1927	CO_2 Lbs. Per Million Gallons	Coke Lbs. Per Million Gallons	Total Alkalinity Settled	Total Alkalinity Carbonated	Total Alkalinity Filtered	Phenol Alkalinity Settled	Phenol Alkalinity Carbonated	Phenol Alkalinity Filtered
September	238	80	55	54	51	28	8	8
October	271	85	57	54	52	32	7	6
November	298	101	64	54	55	38	6	6
December	240	82	64	56	53	33	12	7

Five Miles of Concrete Pavement Laid By Batch-Box Method

Notes on Equipment, Methods and Forces Used on State Aid Job

ONE of the several important state highway jobs now under construction radiating from the city of Chicago is that known as 22nd St., or Route 390. At present, the Chicago Heights Coal Co., Chicago Heights, Ill., contractors, are paving about 5.38 miles of this highway. The work starts at First Ave. and ends at York Road. While not one of the most important highways in the district, it is one widely used by local traffic and acts to relieve congestion on parallel routes. Formerly paved with macadam, it had outlived its usefulness and had deteriorated to the point where maintenance would not be worth while. The new pavement is of reinforced concrete, of standard state section, and will be 20 ft. wide, except at intersections, where it has been widened to 40 ft. The section used has thickened edges, 9 in. thick for 1 ft., and then thinning to 7 in. in a width of 2 ft., which thickness is then maintained to the centerline. The centerline of the slab is divided by a steel center strip. A temperature bar is placed in the thickened edge. The widening at the intersections is done by adding 10 ft. shoulders, separated from the main pavement by steel parting strips of the same design as used in the centerline.

Fig. 1—Final Grading with New Trench Machine and Austin Pup

Rough Grading.—Rough grading, intended to remove old surface and bring the subgrade to within plus or minus two-tenths of a foot of the desired finished subgrade, was done with two elevating grader outfits. For a majority of the work only one outfit was used, while two were employed for a part of the time. The total material to be moved was about 35,000 cu. yd. Balance points were far apart, the average haul being about 1,200 ft. Each grader

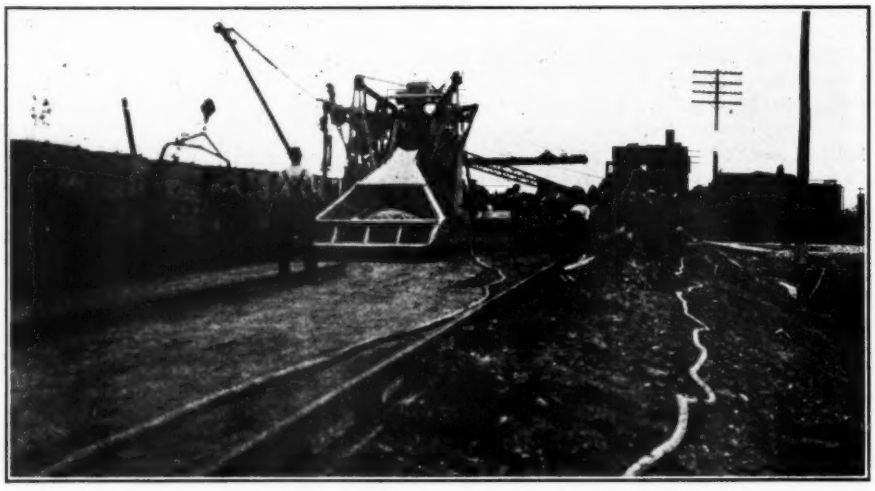


Fig. 2—View of Paver with Boom About to Pick Up Batch Box

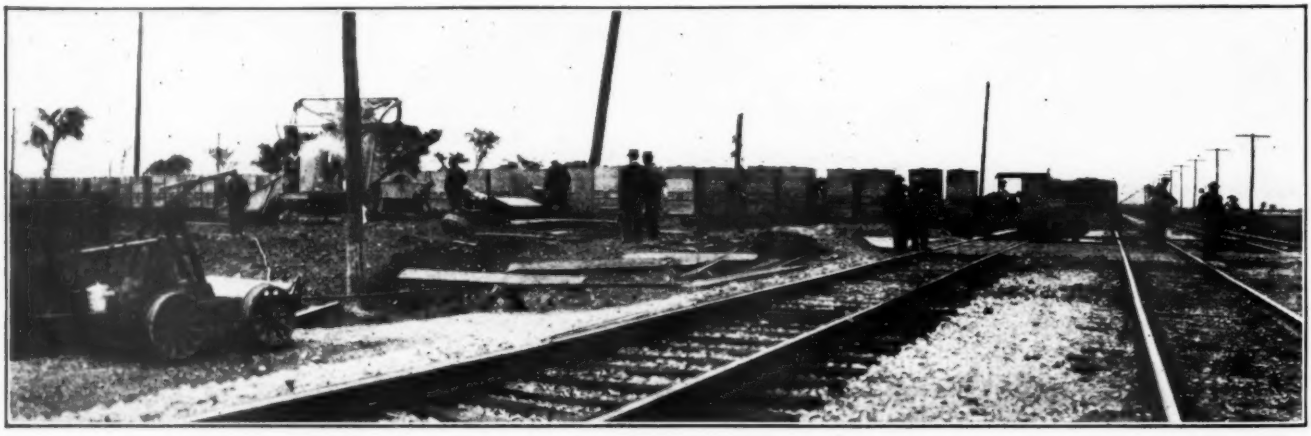


Fig. 3—View of Paving Operation Showing Locomotive, Cars, Batch Boxes, Paver, and Finishing Machine. Note How Tracks Are Blocked Up Over Railroad Crossing, with Man Stationed at That Point to Remove Track When Trains Come Along

outfit, working 1 eight hour shift per day, consisted of an Austin elevating grader hauled by a Caterpillar 60 tractor, 8 dump wagons with three-up team, and a two-up morman team at the dump. The crew then would consist of a foreman, with 9 skinners, a tractor operator, a grader operator, a dump man, a barn boss, a blacksmith, etc. With this organization, the average performance, handling old macadam and clay, was 600 to 700 cu. yd. per day.

Subgrading.—Subgrading was done in a simple manner. The actual cutting and filling was done with an A. W. French "Nu-Method" Finish Grader working on the road forms. This grading was followed with a rolling with an Austin Pup. The finish grader used is a gasoline powered device that plows and blades the subgrade and strikes it off with a template, all automatically. The results on this job were observed to be quite satisfactory, with about the proportion of soft spots that should be expected in this type of soil. Such soft spots as were found could be easily filled with a shovelful of clay ahead of the roller.

Batching Plant.—Located so as to give a maximum haul of batches of about 2½ miles, a batching plant was set up to be served by trucks. This consisted of space for stockpiles, a set of Johnson bins, cement trailers, and an Erie 1 yd. crane for handling the aggregates. Aggregates were trucked from a quarry 1½ miles away by 2½ ton trucks, using 3 on sand on 2 on stone. The sand and stone would then be dumped onto the stockpile, holding on an average 10 cars of each aggregate, and loaded into the bins by means of the crane. A 6-wheel Anderson truck with three trailers was used to haul cement from the cars on a team track about 1½ miles distant. The trailers would hold about 240 sacks of cement in one load. One trailer load would be in transit while one would be unloading to the batch boxes at the plant and the third would be in the process of loading at the car.

Hauling to Paver.—Batches were hauled from the batcher plant to the paver in batch boxes. Two boxes were loaded on each car, of which some were Westerns and some were Lakewoods, and trains were made up of 20 cars, or 40 batches, hauled by 8-ton Plymouth gasoline locomotives. Three such locomotives were used. The track gauge was 24 in. The batch boxes used were those manufactured by Lakewood. This type of equipment proved feasible on this job by eliminating the cutting up of the subgrade by trucks, by keeping a steady stream of materials arriving at the paver, which eliminated waiting for batches, and because traffic tieups were avoided. The contractor was of the opinion that batch trucks would have been more costly to use on this particular job.

Paving Equipment.—The paver used on this job was a new model Koehring 27-E machine equipped with a boom for handling the batch boxes. While being operated by a substitute operator at the time the job was visited, the concrete appeared satisfactorily uniform and the mixing cycle was averaging 80 seconds with a specified one minute mix. Lost time was not excessive, since a good supply of batches was on hand at most times, and the performance was reported to be about 400 batches or 800 lin. ft. per shift with the regular operator. The lost time observed was mostly due to shifting the paver and to waiting for an empty train to pull back to the passing track and a full load to pull up to the paver. This passing track was kept as near to the paver as possible in order to minimize this lost time. The mix used was 1:2:3½, with enough water to give a slump of less than 2 in. At the intersections 3 additional sacks of cement were added to each batch in order to secure normal strengths at 7 days instead of 28 days in order that the intersections would not remain blocked for very many days.

Pipe Line.—Water for the batches and for other purposes was taken from the Des Plaines River and pumped to the paver through a 2 in. pipe line by a 500 g.p.m. 3 cylinder C. H. & E. gasoline motor driven pump. This pipe line was found to give plenty of water, since no great amount was needed for curing.

Finishing.—As rapidly as it was deposited on the subgrade and spread by the puddlers, the slab was finished with an Ord finisher and the usual floats. As rapidly as finished, the slab was covered with burlap and this covering wet down thoroughly and kept wet by sprinkling. This covering was kept in place until the following morning, when it was removed and calcium chloride was spread over the surface at the rate of 2½ lb. per square feet.

Subgrading Crew.—The crew on the rough grading work has already been enumerated. The fine-grading crew consisted of

1 Superintendent
1 Timekeeper
1 Subgrader operator
1 Roller operator
2 Form setters
2 Laborers

The Batcher Crew.—Men at the batcher plant and hauling materials and batches, were as follows:

6 Truck drivers
2 Cement hands at car
4 Cement hands loading boxes
1 Crane operator
1 Batcher
1 Fireman, on crane
1 Checking loads
3 Locomotive operators

The Paving Crew.—At the paver and in the finishing operation, the following men are used:

1 Foreman
1 Mixer operator
1 Finishing machine operator
1 Hoist man
2 Dumpers
2 Finishers
3 Puddlers
1 Laborer setting steel
1 Water boy

Supervision.—Supervision on jobs being done by the Illinois State Highway Department is quite thorough. Besides setting grade stakes and generally supervising the work, the project engineer keeps accurate count of the proportioning at the batchers, the number of batches sent to the paver, the time of arrival and departure of batch trains, the time of dumping, the time of start and end of mixing, the time of dumping the batch, lost time and cause, water

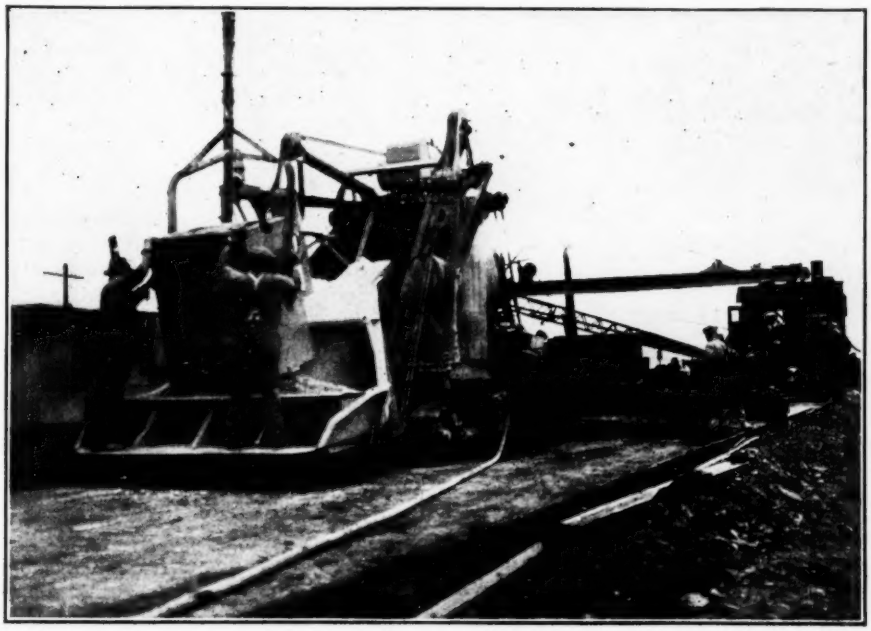


Fig. 4—Two Men Are Required to Dump Batchers in the Charging Skip. The Locomotive Crane in the Background Is Being Used by the Railroad in Reballasting the Track at the Crossing

Fig. 5—Curing Is Done with Wet Burlap and Calcium Chloride. Burlap Is Handled on Batch Cars as Shown

content of the mix, evenness of surface, and other pointers. He takes frequent slump tests and specimens for beam tests, and otherwise observes everything that goes on at the work. Strict adherence to specifications is enforced, and careful record of performance is kept in order that the engineer may be satisfied that the work is progressing efficiently. As a result, the work such as that described is well done.

The contractor on this job is the Chicago Heights Coal Co., of which W. J. Orr is secretary and in charge when the work was visited. The state, which pays for the entire job and furnishes the cement, is represented by George Lamb, district engineer; Oscar (Jack) Frost, resident engineer; George Squires, engineer of construction; R. M. Dillon, supervising engineer; and Norman Saper, junior engineer.

Rate of Deformation of Concrete During Test in Compression

From May Technical News Bulletin

The rate of deformation of materials during strength test affects the strength values that are obtained, an increase in rate of deformation resulting, in general, in an increase in strength. This characteristic is apparently connected with the flow or the rate at which the particles of the body will readjust themselves to new positions during strain and is an entirely different phenomenon from the influence of the inertia of the mass of a structure which will permit a much higher instantaneous loading, such as will result from an explosion, than will be statically sustained. Tests by Abrams, described in Proceedings of the American Society for Testing Materials, Vol. XVIII, 1917, p. 364, have shown the relation between the strength of various concrete mixes and the velocity of the testing machine head, the actual deformation of the concrete itself not being given.

In connection with the tests on the Santeetlah Dam cylinders, described in Technical News Bulletin No. 129 (January, 1928), several of the 6 in. cylinders were tested with plaster of Paris caps on both ends, and several were tested without capping, the ends having been previously ground to a true plane surface. The idle speed of the testing heads of the machines was 0.05 in. per minute, generally adopted as standard, and the requirement of A. S. T. M. specification C39-25. These latter machines are equipped, in accordance with the foregoing specification, with a spherical bearing block for the top of the specimen.

The results obtained are tabulated below. Although not shown in the table, the rate of deformation is very slow at first, then increases in value, becoming practically constant and remaining so throughout the greater part of the test. This latter rate has been given in the table. Although it is to be expected that the rate of deformation of the concrete will be less than the speed of the head when the cylinder is capped on both ends with plaster of Paris, the large difference in speeds is surprising, moreover, in view of the difference in speeds being maintained through the greater part of the test. The speed difference of head and rate of deformation of the concrete cylinder without caps is even more unexpected.

Rate of Deformation and Testing Machine Head Speeds

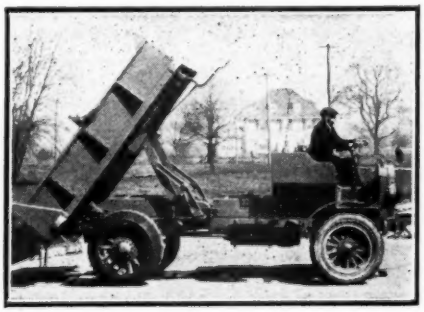
Capacity of machine (in pounds)	Cylinder diameter In.	Speed of testing machine head In. min.	Rate of compression of specimen In. ft. min.
600,000	18	[1]0.050	[2]0.0019
			[3].0021
	12	[1].050	[2].0028
			[3].0035
		[1].055	.....
		.038	[3].0038
	6	.044	[3].0036
200,000		.018	[3].0061
		.021	[3].0074
		.022	[3].0070

[1]Idle speeds.
[2]Specimens capped with plaster of Paris on both ends.
[3]No caps; cylinder ground on both ends.

New Hoist Mechanism Gives 60° Dumping Angle

The Autocar Co., truck manufacturers of Ardmore, Pa., has just perfected a new rotary dump hoist mechanism which it is stated will raise the body to an angle of 60 deg.

The Autocar rotary dump mechanism is one of the oldest power dumping



Autocar With New Rotary Dump Hoist Mechanism

mechanisms on the market. It was developed by the Autocar Co. in the early days of the motor truck industry in response to a demand for a method by which the power of the engine would be used to elevate the dumping body instead of hand power dumping with a crank.

One of the most important features of this new mechanism is the automatic cut-out which, as the name implies, cuts off application of the power just as soon as the body reaches the maximum elevation and similarly cuts off the power when the body has returned to full normal position. This requires no attention from the driver of the truck and he may put the truck in motion before the body has returned to normal.

The new 60-deg. dumping angle will greatly increase the ease with which a wet-batch may be dumped and it is chiefly for the highway and construction trade that Autocar perfected this new mechanism.

A $250,000 Per Mile Interurban Highway in Scotland

Heavy Construction on 42-Mile Road Between Glasgow and Edinburgh

By C. L. McKESSON
Materials and Research Engineer, California Division of Highways

THIS very unusual highway project 42 miles in length extending from Glasgow to Edinburgh in Scotland is of particular interest in that it is probably one of the most expensive pieces of road construction ever undertaken in Great Britain that it has been carefully designed to meet the designs of modern high speed interurban traffic, and that in type of construction it exemplifies the most approved European construction.

Cost and Quantities.—Some idea of the magnitude of the project will be conceived from the following table of costs and quantity: The total estimated cost is $10,878,262, including $1,776,272 for bridges and $942,962 for lands. The average construction cost, exclusive of bridges and right of way, is about $195,000 per mile. Excavation soft material 2,150,000 cu. yd., excavation rock 67,000 cu. yd., weight of Telford base 300,000 long tons, weight of surfacing 153,000 long tons, length of drains 82½ miles, maximum depth of cut 24 ft., maximum height of embankment 44½ ft.

On Relocated Alignment.—The road traverses a rather sparsely populated section and passes through several villages but no large towns or cities. Most of the road is on new location although old routes have been utilized for a portion of the way. Coal mining has been carried on for many years in the region traversed by the road and subsidence of the entire country side occurs from time to time. This unusual condition has been taken into consideration in the design of bridges which have been made adjustable by keeping the deck separate from the abutments in such places and by providing recesses in which hydraulic jacks can be placed for the purpose of raising the bridge deck from time to time as this becomes necessary. In one or two places where the coal has not already been mined the highway authorities purchased the coal "in place" from the owners with the understanding that it be allowed to remain and that lowering of the road be thus avoided.

Separate Traffic Lanes.—In the design of this improvement the engineers took into consideration the probability of urban development along the new route. They also took into consideration the difficulty which has been encountered in Europe and elsewhere in getting traffic to utilize to the best advantage a roadway containing a large number of traffic lanes. Experience on some very wide pavements near London had indicated that it is very difficult to secure a distribution of traffic on excessively wide roadways. As a result of their observations the engineers who designed this work have planned that the entire improvement ultimately have two traffic lanes each 30 ft. in width.

Details of Design.—On approximately four miles adjacent to Glasgow

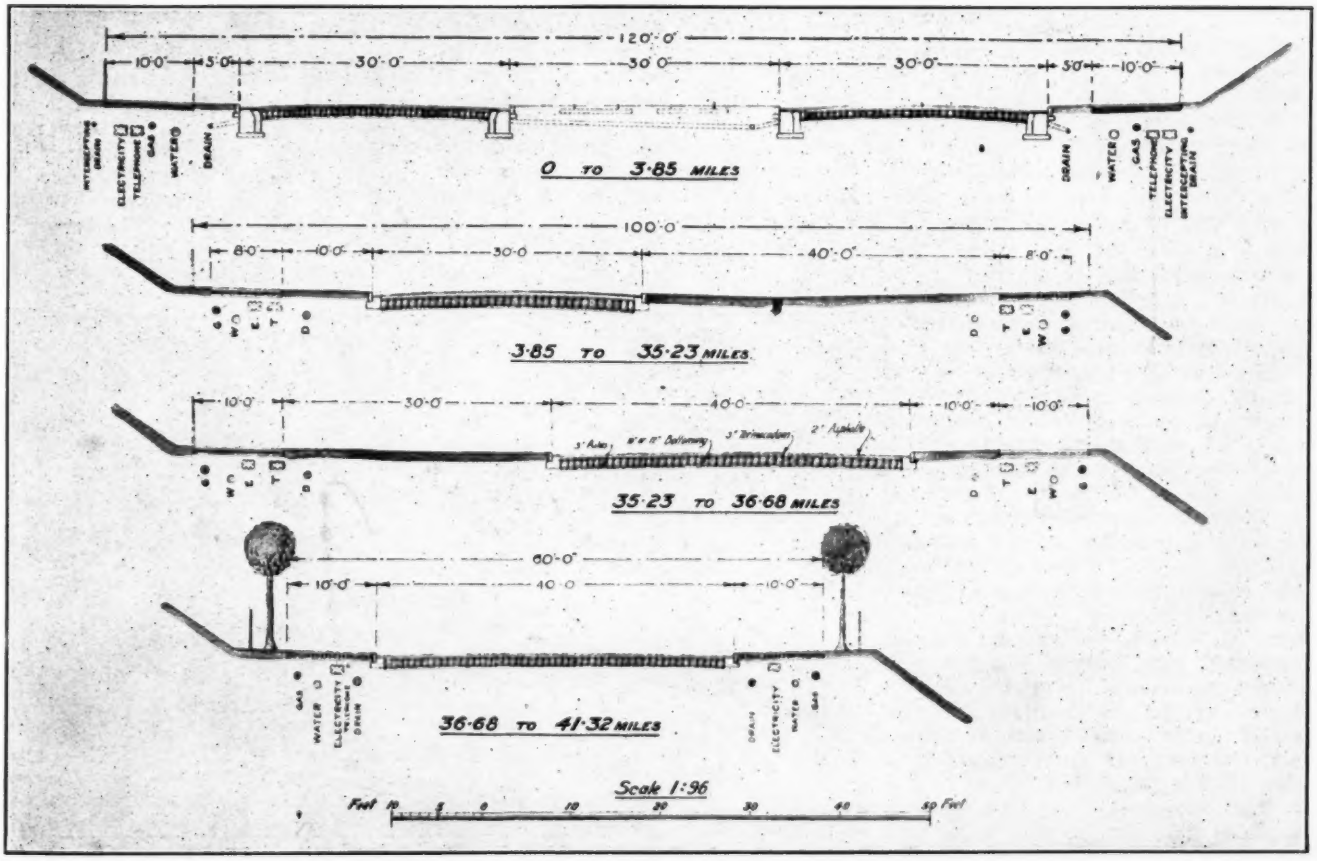


Fig. 1.—Typical Cross Section of the 42-Mile Interurban Highway Now Under Construction Between Glasgow and Edinburgh

Fig. 2.—Telford Base Under Construction on Glasgow-Edinburgh Highway

the roadway has been graded to a width of 120 ft. between slopes. The typical cross section shows the 10-ft. sidewalks, a 5-ft. parking on each side for service pipes, two 30-ft. roadways with a strip in the center 30 ft. in width which it is expected may be used by an electric railway. On the next 31 miles the road is being graded to a width of 100 ft. between slopes and one 30-ft. roadway is constructed with one edge adjacent to the center line. The cross section also shows an 8-ft. sidewalk. On this section it is planned that future development, when required, will include the construction of another 30-ft. roadway separated from the first by a curb along the center line of the road. Figure 1 shows the typical sections and also indicates the type of construction.

In general, the road is to have a Telford foundation about 12 in. in thickness placed on a cinder sub-base.

Figure 2 shows this telford base under construction. A very heavy concrete curb has been constructed along each side of the roadway extending to the full depth of the telford base. This curb serves as a foundation for a 6 in. by 12 in. stone curbs which projects above the roadway with a 6 in. curb face. Figure 3 shows a section of the finished roadway on the Glasgow end of the project. After the telford base is thoroughly locked and filled the road is surfaced with 3 in. of tar macadam which is to be followed, after traffic has used the road for a time, by a 2 in. covering of asphalt concrete. This type of construction appears massive compared with some of our thin reinforced road slabs but is, in general, typical of the practice that has prevailed on important roads in France and Germany for many years. It is due to this heavy but more or less flexible base that it has been possible to build up road surfaces which successfully withstand continuous traffic, practically without load limit. It is nothing unusual to see heavy steam driven trucks with net loads of 30 tons or more traveling at high speed.

The surfacing on this road, first of tar macadam and then of asphalt concrete, is also more or less typical of English practice.

On some of the new grade reinforced concrete base is being constructed. This base has a cinder and stone subbase and is itself in reality a very substantial pavement. It is laid in 20-ft. panels with reinforcement top and bottom. The panels are constructed alternately and a reinforced concrete beam is built under each transverse joint. The concrete is 8 in. in thickness and of 1:2:4 mix. This very rugged concrete pavement is to have a cover consisting of 2 in. of asphalt concrete. It will be noted that the total thickness including sub-base reinforced concrete and asphalt wearing surface is 18 in.

Storm Drains.—A storm drain system is being installed and catch basins provided throughout the entire route for handling storm water. Considering the great width of the improved roadway and the heavy type of construction it is not at all surprising that the total cost of this road should reach the stupendous average of $250,000 per mile. On the section adjacent to Edinburgh the total estimated cost is $369,854 per mile.

Intersections. — The chief engineer has given much attention to the proper design of intersections and also to the matter of securing proper sight distances on vertical curves. Figure 4 shows the standard solution which he has applied to the intersection problem. The intersections, where completed, under this plan had a very pleasing appearance and seemed to insure reasonable sight distances.

The method of using the standard solution shown in Fig 4 is as follows:

The center of the 30 ft. carriageway and of the intersecting road are drawn and are found to intersect at the point A. With A as center and a radius equal to four times the width of the carriageway of the intersecting road (minimum 70 ft.) the "locus circle" is drawn on which four points B. B. B. B. are found equal distant from the curbs of the 30 ft. carriageway and of the intersecting road. With these point as center the curbs are swept in. If the angle of intersecting is such that any of these corners are greater than about 135° the radius of the "locus circle"—in these corners only—is increased by 10, or by 20 ft. if greater than about 150°.

To ascertain the splay angle on the fences a point C is found on the center line of the intersecting road at a distance square off line of the new road fence not less than F, the distance between the fences of the intersecting road. The distance Y between the point C and the point D where the center of the intersecting road crosses the line of the new road fence is set off right

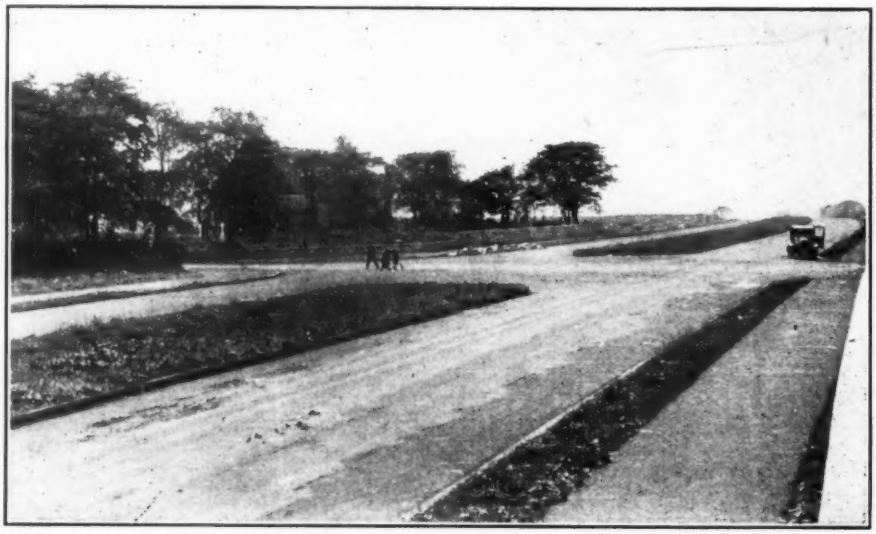


Fig. 3.—Section of Finished Roadway on Glasgow End of Project. The Sidewalks Are of Tar Macadam

and left along the fence, thus giving the points E. E. The splay required lies along the lines Ce CE.

In every case, of course, it is necessary to arrange the position of C so that the splay is set far enough back not to reduce the width of the footpath.

Sight Distance on Vertical Curves.—The following is quoted from the chief engineer's description of the design of the vertical curves:

"Vertical curves at sags are to be parabolic. The length of the curve is to be generally 150 times the algebraic difference of the grades per cent (equivalent to a circular curve of 15,000 ft. radius). Vertical curves at summits are to be parabolic. The length of the curve is to be 120 times the algebraic difference of the grades per cent (equivalent to a circular curve of 12,000 ft. radius) except at bridge or other crossings where it is necessary or desirable to keep down the amount of banking, where a length of 80 times the algebraic difference of the grades per cent is desirable and a length of 50 times will be the ultimate minimum (equivalent to a circular curve of 8,000 ft. and 5,000 ft. radius, respectively). With such vertical curves at summits, the minimum lengths of the line of sight to the horizon from a point in a motor car 5 ft. above the ground are respectively 346.4 ft., 282.8 ft., and 223.6 ft., and the distances at which two motor cars 5 ft. high become mutually visible are 692.8 ft., 565.6 ft., and 447.2 ft."

This important highway project is conducted under the general supervision of Sir Henry Maybury, chief engineer of transport. The proposal of plans, details, and the construction of the work is carried on under the personal direction of Mr. D. A. Donald as chief engineer. It is to these gentlemen and to Mr. Wright, assistant engineer, that the writer is indebted for the information which he obtained regarding the project and for the opportunity to make this inspection.

Acknowledgment.—The matter above is taken from a paper presented on March 10 by Mr. McKesson at a meeting of the Western Association of State Highway Officials.

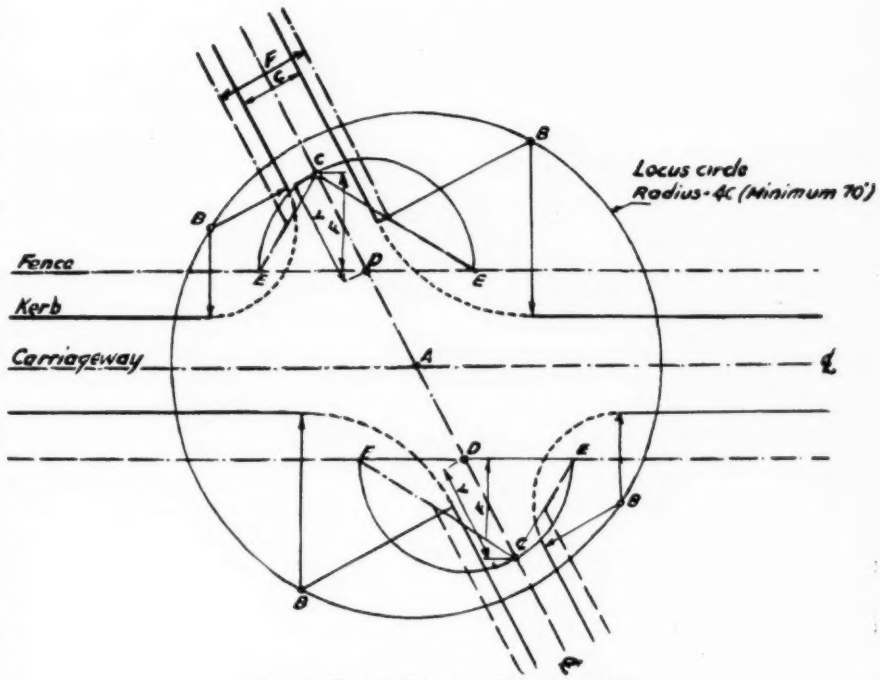


Fig. 4.—Typical Layout of Road Junctions

Cost of Blade Grading Work

Mr. Andrew Wallubait, Assistant County Engineer of Dallas County, Iowa, states, the costs listed in the following table are operating costs only and do not include costs of tractors or any depreciation whatsoever. The county is divided into three districts in charge of one of the supervisors. Each crew works in its own district. The operators are paid a flat rate of $0.65 per hour for actual working time only, grading, moving and repair, etc. They furnish their own transportation and pay for their own mess.

All the grading is staked by the engineer who has charge of all the crews. Where team work is necessary this is done first and followed up by the machine. The roadbed has been made 22 to 24 ft. depending on the importance of the road. Nearly all the work is back-sloped and left in good shape. The repairs listed under Tractors Nos. 2 and 4 in Crews Nos. 1 and 2, were spent before the work opened up and a short time later these tractors were traded in on the new tractors, Nos. 1 and 3 as shown. Crew No. 1 graded about one mile with Tractor No. 2 and Crew No. 2 graded about two miles with Tractor No. 4 before the new tractors were shipped. Crew No. 3 used Tractor No. 5 all season as shown.

The operators and all repairs, etc., are paid for out of the County Road Fund and then the Townships are charged a flat rate of $4.00 per hour for the actual time spent on grading only.

The miles shown under each crew are the actual miles upon which they worked. Crews 1 and 3 had the better part of the county while Crew No. 2 had the more hilly part and more banks to fight.

Blade Grading Costs, 1927, in Dallas County, Ia.

Items	Crew No. 1 Harper & Campbell Tractor No. 1	Crew No. 1 Harper & Campbell Tractor No. 2	Crew No. 2 Woods & Whitney Tractor No. 3	Crew No. 2 Woods & Whitney Tractor No. 4	Crew No. 3 Randall & Sieberry Tractor No. 5	Totals, hours
Hours of Labor						
Twp. grading	2,160		1,849		16,888	5,697
County grading	60		270		70	400
Moving	369½	40	258	70	144	881½
Engine repair	117	306	385	385	436	1,629
Blade repair	175		227		135	537
Total, hours	2,881½	346	2,989	455	2,473	9,144½
Grand total, hours		3,227½		3,444	2,473	9,144½
Parts						
Tractors	$ 58.32	$432.48	$ 62.09	$154.08	$ 163.19	$ 870.16
Graders	188.77		575.40		309.94	1,074.11
Gasoline						
Gallons	6,219		6,368		4,840	17,427
Per gallon	$ 0.162		$ 0.148		$ 0.146	$ 0.153
Total	$1,012.82		$ 947.47		$ 710.95	$ 2,671.24
Oil	$ 169.10		$ 85.27		$ 139.58	$ 393.95
Grease	$ 36.20		$ 14.40		$ 25.15	$ 75.75
Waste	$ 4.95		$ 5.28		$ 2.20	$ 12.43
Total	**$4,000.51**		**$4,082.59**		**$2,958.46**	**$11,041.56**
Miles graded	44*		34*		31*	109
Cost per mile	$ 90.92		$ 120.07		$ 95.41	$ 101.30

*Crew No. 1 backsloped all but 4 miles. Crew No. 2 backsloped all but 2 miles. Crew No. 3 backsloped all but 2 miles.

Road Construction in Argentina.—A recent report of the U. S. Trade Commission at Buenos Aires states that Argentina spent $5,437,700 on road and bridge construction and maintenance in 1927 out of national funds. It is estimated that an equal amount was expended by the provinces from their individual provincial budgets.

The Gasoline Tax in Mexico.—The Mexican federal government's road tax on gasoline is said to have produced $7,500,000 U. S. currency during 1927, and about $5,000,000 of this amount is said to have been set aside for work during 1928 on the road from Laredo, Texas, southward in the direction of Mexico City.

The Design of Settling Basins for Sewage Treatment Plants

"Floccular" and "Granular" Sludge; Velocity, Time and Surface

By DR.-ING. KARL IMHOFF
Chief Engineer, Ruhr Verband, Essen, Germany
Translated by ARTHUR B. MORRILL, Assistant Engineer Filtration, Department of Water Supply, Detroit, Mich.

IDEAS regarding the design of settling basins for sewage treatment have often changed in the course of time. Even today there are no generally recognized rules. The first designs were based on the observation that there is a smaller velocity in the settling basin than in the conduit which conveys the sewage. Hence, it was believed that the amount of sedimentation effect could be calculated from the velocity of flow, and it was assumed that the lower the velocity, the greater the efficiency would be. Then however, it was observed that the velocity was not the true criterion as soon as a certain lower limit was passed, and that below this limit all basins worked alike, quite without distinction, whatever the velocity of the water might be.

Then in the "Technisches Gemeindeblatt" of August 20, 1903, Steuernagel proposed for the first time to design settling basins on the basis of the "flowing-through-time." Steuernagel proved that for the types of basin construction then prevalent, effectiveness was really a function, not of the velocity, but of the time. This method of design was gradually adopted in practice.

Then there appeared in the "Technisches Gemeindeblatt" of Dec. 5, 1918, an article by Schulz of Prague, who maintained on the basis of experiments and physical considerations that design on the basis of time was erroneous, and that settling basins should be designed only on the basis of surface area. This amounted to saying that the effectiveness of a settling basin depended not on its volume but on its surface area, and that the depth of the basin was quite immaterial.

Schulz's contention contradicted experience with municipal sewage. The contradiction was to be explained by the fact that Schulz had made his experiments, not with municipal sewage, but with soil washings from sugar beets. In his physical analysis Schulz assumed that the settling velocity of the separate particles of suspended matter remained constant as they settled, right to the bottom of the basin. This assumption may be correct for earth washings, but it is not true for municipal sewage, for there the separate suspended particles are more or less flocculent and have the property of agglomerating or collecting together into larger particles as they settle. Thus the suspended particles become gradually larger with increase of depth and their velocity also increases, for Schulz agrees that the settling velocity of the suspended matter depends on the size of the particles. Hence a distinction must be made between floccular and granular suspended matter, then the existing contradiction between Schulz and Steuernagel is explained. Settling basins for sewage having floccular sludge must be designed by Steuernagel's method (according to time), and for sewage with a granular sludge by Schulz's method (according to surface area).

Design on the basis of velocity is important only in special cases, as for grit chambers.

Grit Chambers.—Grit chambers preceding purification plants are necessary only for large installations and combined systems. They should have flat bottoms, not hopper shaped, in order that sludge may not be mixed with the sand. The bottom may be provided with a drain, closed while the water is flowing through the chamber, in order that the sand may be drawn out in a dry condition, after the flow has been cut off. The cross section of the grit chamber (width times depth) is so determined that the average velocity of the water is one foot per second. This scouring velocity is necessary in order that only sand but no sludge shall be

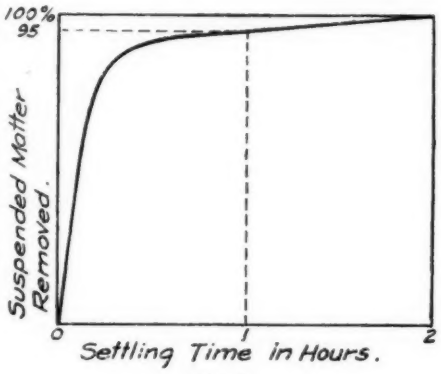


Fig. 1

deposited. Thus the following formula applies:

$$\text{Cross Section (sq. ft.)} = \frac{\text{Flow (cu. ft. per sec.)}}{\text{Velocity (1 ft. per sec.)}}$$

The length depends on the volume of sand that it is desired to accumulate before cleaning. The longer the better. Two or three interchangeable chambers and a by-pass are necessary. In large plants grab buckets are used for cleaning.

Settling Tanks.—In sewage a definite volume of the sludge material separates out in a definite time. The process is exactly the same in a settling plant as in a (Imhoff) settling glass, except that in the latter case the sewage does not remain motionless, but is flowing slowly through the plant. (It is only necessary to imagine that a full settling glass is floating with the rest of the water through a clarification basin.)

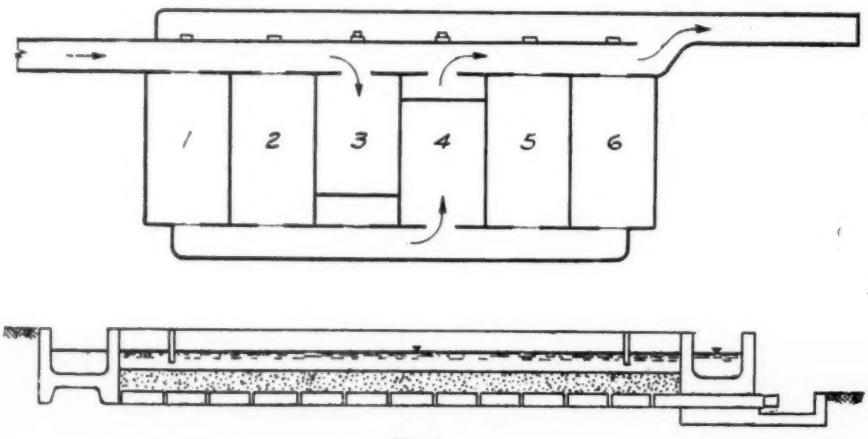


Fig. 2

The operation of a continuous-flow settling plant, for equal depths, is exactly the same as in a settling glass, assuming that the flowing-through time in the settling plant is just the same as the time of standing in the glass. The velocity of flow under these conditions is, with horizontal flow, quite immaterial as far as the clarification is concerned, as long as a certain limiting velocity is not exceeded. That is the velocity at which the sludge particles which settle out no longer remain lying on the bottom, but are rolled along. However, this permissible limiting velocity is so great (surely more than 10 ft. per minute) that under ordinary conditions it never can be reached.

The only exception is in the case of the newly devised flat "underdrained settling basins" (described further on) that have water depths of only 4 to 16 in. For these basins it is always necessary to determine, by the following formula, whether the permissible limiting velocity has not been exceeded:

$$\text{Velocity (ft. per min.)} = \frac{\text{Length (ft.)}}{\text{Time—(min.)}}$$

In the case of ordinary deep tanks, for example, Imhoff tanks, the velocity never has to be considered. Much less can it be used as a basis of design.

The customary basis of design of a settling tank, and according to the above argument the proper one, is the flowing-through time, according to the following formula:

Settling Volume (cu. ft.)=Flow (cu. ft. per hr.)×Settling Time (Hrs.)

By settling time above is understood the theoretical, average flowing-through time, which is calculated inversely in the case of any given basin by the following formula:

$$\text{Settling Time (Hrs.)} = \frac{\text{Settling Volume (cu. ft.)}}{\text{Flow (cu. ft. per hr.)}}$$

If, on the basis of this, it is desired to design a settling basin for a given sewage, the settling curve up to two hours is plotted on the basis of investigation in the settling glass (Fig. 1). Assuming that the curve shows in one hour an efficiency of 95 per cent of the settleable material, and that this efficiency is considered sufficient, it is only necessary to design the settling space of the basin, according to the above formula, on the basis of a flowing-through time of one hour.

This design presumes, however, as above said, that the average depth of the basin is the same as the depth of the settling glass in which the sewage was investigated. That, however, is usually not the case. The settling glasses are 16 in. deep, while most basins have an average depth of 5 to 7 ft. If it is desired to take account of this condition, the effect of depth must be investigated. Here it is necessary to distinguish between granular sludge, in which the individual particles of suspended matter settle to the bottom with uniform velocity and independent of each other, and floccular sludge, the particles of which in settling continually gather into larger flakes and consequently continually increase their velocity of settling. Examples of granular sludges are coal sludge or sugar beet washings. Examples of floccular sludge are precipitated aluminum sulphate or activated sludge. Municipal sewage sludge stands somewhat between the two, inclining however, rather more to floccular sludge.

For purely granular sludge, the laws proposed by Schulz of Prague apply, that is, the depth of the basin is quite immaterial. There is thus no point in considering either time or volume. The only criterion is surface area. This is determined by the following formula:

$$\text{Surface Area (sq. ft.)} = \frac{\text{Flow (cu. ft. per hr.)}}{\text{Min. Settling Vel. (ft. per hr.)}}$$

Minimum settling velocity means here that velocity which (in the above case) the last of the desired 95 per cent of the suspended matter must have had in order to reach the bottom of the 16-in. deep settling glass. Thus the minimum settling velocity of 95 per cent of that part of the suspended matter settleable in two hours=the height of the settling glass (16 in.) divided by the settling time (1 hr.)=16 in. divided by 60 minutes=0.267 in. per minute.

Where the sludge is granular, settling chambers with vertical upward flow of the water (such as the obsolete settling tanks and towers) are also designed solely on the basis of surface area. Their upward velocity of flow must be not greater than the above calculated minimum settling velocities. Great depths of such structures are therefore useless. The most favorable are very shallow basins with the greatest possible surface area, such as flat under-drained settling basins.

For floccular sludge the case is exactly the opposite. Deep basins are much more favorable than shallow basins for the same surface areas. Also motion of the water in a vertical direction upward is of advantage in that the sewage passes through a sort of floating filter. Unfortunately the various kinds of sludge are so different that it is not easy to formulate a general rule such that, from study of floccular sludge in the settling glass, the operation of settling basins of quite different depth can be predicted. To be quite certain in the design, it would be desirable to make the depth of the settling glass just the same as that of the proposed settling basin (as Steuernagel did at Cologne 20 years ago) and then calculate the volume required from the flowing-through time, which is made equal to the settling time in the settling glass.

All settling basins should have a skimming board at the inlet and at the outlet, and in addition a long overfall weir at the outlet. In the case of putrescible sludge, the principal thing is that it must never remain in the basin so long that it is again stirred up by the gases of decomposition and can be carried away in the effluent. According to the manner in which the sludge is removed from the basin, the following types of basin are to be distinguished.

1. **Ordinary Settling Basins.**—The sludge remains lying under water until it begins to rise. After a week or two, the entire basin is emptied, cleaned and refilled. In the design of continuous flow basins, the volume to be occupied by sludge is to be deducted. Furthermore, at least one basin is to be deducted as reserve.

2. **Hopper-Bottomed Basins.**—The bottom is divided up into hoppers from the bottom of which sludge is removed about every two days, without interrupting the operation. The entire volume of the basins may be considered effective after deducting the volume of the sludge.

3. **Basins with Cleaning Machines.**—The bottom of the tank is nearly horizontal and the machine is movable (Fidler scrapers, Dorr thickeners, Hurd endless chain scrapers). The volume occupied by the sludge must be deducted in the design.

4. **Earthen Lagoons.**—These are only possible with non-putrescible sludge. The sludge remains lying where the water has deposited it.

5. **Flat Underdrained Settling Basins (Fig. 2).**—These are arranged like sludge drying beds, except that the sewage flows over them. The drainage system can be closed and it is kept closed during the operation of sedimentation, and only opened when the basin is put out of service and the sludge is to be dried. Thus each basin operates alternately, first as a settling basin and then as a sludge drying bed. It is desirable to operate always with two basins in series in order that the first basin may accumulate as much sludge as possible while the second takes care of satisfactory clarification. In order to give time for the drying and removal of sludge, there must be provided at least three times as many basins as is indicated by the design on the basis of the volume of water. In Fig. 2, for example, basins 3 and 4 are in operation while the other four stand empty. For coal sludge, such basins are made from 6½ to 13 ft. deep. For municipal sewage sludge, however, they are only possible for very small depths of from 4 to 16 in.

The sludge is removed in a dry state from such basins. Because of their moderate cost and their simplicity, they are to be preferred as preliminary clarification plants in cases where the drainage of a city is still in a process of development. If later, on complete development of the drainage system, they begin to give trouble from odor, they can be used further as sludge dry-

ing beds for a final clarification plant (for example, Imhoff tanks).

6. **Two Story Settling Basins.**—The sludge space is provided under the settling basin proper. The sludge slides automatically and continually out of the upper space and into the lower. This is of advantage only when the sludge is putrescible. Then the lower chamber becomes a digestion chamber. The sewage flows through the settling chamber and is kept in a fresh condition. The sludge, however, passes into the digestion chamber and there decomposes to such an extent that it becomes odorless and dries easily upon the sludge drying beds. In the design the entire settling space is to be considered effective.

R. D. Wood & Co. Celebrate 125th Anniversary

The well-known firm of R. D. Wood & Co. of Philadelphia, Pa., this year celebrates its 125th anniversary. This firm played an important part in the early development of the United States through its activities in the construction of water works and gas plants. A brief history of the firm and its activities will be of interest.

In 1803, David C. Wood, uncle of Walter Wood, the present owner of R. D. Wood & Co., purchased a large tract in southern New Jersey on the Maurice River at Millville, where a blast furnace for smelting ores and operated by water power was erected. Fuel was supplied by the timber on the tract and iron ore was brought from the bog deposits on the Delaware Bay. The location of the plant was unique in that water power, tide water, fuel and raw materials were all obtainable in one locality.

In addition to the metal, there were cast stove plates, fences, lamp posts and iron water pipes—the first to be cast in America—for the City of Philadelphia Water Works.

During the following few years were erected saw mill, grist mill, glass house, cotton mill, and bleachery, and in 1856 an additional foundry—one of the first in the United States for the making of cast iron pipe in an upright position.

In 1867 additional foundries were acquired at Florence, N. J., and in 1863 the Starr Foundries at Camden, N. J., were purchased. Both these plants were in subsequent years considerably enlarged. In 1900, operations at the original Millville Foundries were suspended, and the business transferred to the larger and more modern works at Florence and Camden.

In 1906, the Florence Works were again enlarged, and then in relatively rapid succession came the disposal of the Camden plant, and in 1926 at the Florence plant the erection of another great foundry for the manufacture of "Sand Spun" Pipe, which is cast centrifugally in refractory molds.

Today at Florence, N. J., on Delaware River tide water, and under one management, is a great plant covering 54 acres, employing approximately 1,500 men, engaged in the manufacture of cast iron pipe, fire hydrants, gate valves, hydraulic machinery, gas machinery, gas producers, etc., and from this plant shipments are made by both rail and water to many parts of the world.

Among the most prominent works executed by this firm are the original iron pumps and turbines which succeeded the undershot water wheels that had long been used by the water works of the City of Philadelphia for lifting water into Fairmount reservoir. Subsequently, they erected the centrifugal pumps used in supplying new filter beds for the City of Philadelphia at Torresdale.

The firm has erected one of the largest single units for steam pumping machinery in the United States. This machinery is still in operation at the California Station in the City of Cincinnati, and has a daily capacity of 125,000,000 gal.

Thousands of tons of materials from this firm, such as water pipe, fire hydrants, gas holders, etc., have been used in the construction of water and gas works on the Pacific Coast. Most of this material was shipped direct from the company's private wharves by steamer, and in the old days, sailing vessels. Before the construction of the Panama Canal these vessels, of course, sailed around the Horn, where frequently bad weather was encountered and many cargoes jettisoned, and some altogether lost.

It is of interest to note that while this firm manufactured the first cast iron water pipe made in the United States, using the very oldest method of casting water pipes in sand molds in a horizontal position, they have just recently completed an entirely new shop for the manufacture of pipe by the newest method of casting pipes by centrifugal force in refractory molds, called the "Sand Spun" process.

Foundries and Works of R. D. Wood & Co., at Florence, N. J.

Water Works and Sewage in Missouri.—All of the 94 Missouri cities with a population of 2,000 or over have water supply systems, and all except four have sanitary sewerage systems, according to a survey made by the Missouri Commission on Public Utility Information. Only five towns in the state with populations more than 1,500 lack water systems. Fifteen such towns lack sewage facilities. There are 229 communities with water-supply system, the surveys shows. Municipal plants serve 156 of these communities, and the remainder depend on privately owned plants. Of the 188 towns with between 500 and 1,000 population, 57 are equipped with water-supply plants, and 28 have sewerage systems. Twelve communities with less than 500 and six with less than 250 population have such facilities.

Water Rates 1876 and 1927 in Danvers, Mass.—The present minimum metered house rate at Danvers, Mass., is exactly one half that of 1876, according to the 1927 report of Roger W. Esty, Superintendent of Water Works. The report gives the following interesting table showing the rates from 1876 to 1927:

Minimum Household Rates with Kitchen Faucet and Bathroom	
1876—1901	$16.00 Fixture Rate
1901—1923	10.50 Fixture Rate
1923—1927	12.50 Fixture Rate
1927	8.00 Meter Rate

The New Bloomington, Indiana, Water Works

An Experience in Water Works Development in Average City

By PAUL HANSEN
Pearse, Greeley & Hansen, Engineers, Chicago

THE new water works of Bloomington, Indiana, are nearing a stage of completion which may warrant a short article covering a number of experiences encountered in the development of the works, which experiences by the way illustrate some usual and unusual factors encountered in water works practice.

The original water works of Bloomington were installed in 1892 as a result of a franchise granted to the Bloomington Water Works Company. This company developed a supply from what is known as Spring Branch several miles to the south of the city. In 1894 the works went into the hands of a receiver and so remained until 1898. In that year the City Water Works Company was incorporated and took over the water works under an agreement whereby the city had the right to gradually purchase all the shares of the company and this was accomplished in 1902.

Spring Branch Development.—The Spring Branch development comprised an earthen dam which failed to retain the water satisfactorily due to crevices in the limestone sides and bottom of the valley. Here it may be noted that Bloomington lies in an unglaciated area of southern Indiana and bed rock lies close to the surface. In 1899, the works were forced to shut down on account of lack of water. In 1901 an attempt was made to impound more water by raising the dam 12 ft., making a total height of 25 ft., and puddling portion of sides and bottom to prevent leakage, but continued leakage under and around the dam prevented this from being effective. Shortages occurred in subsequent years and these shortages became acute following the installation of a sewerage system in 1904 and 1905.

In the latter year and without technical advice the city built a dam 25 ft. high on Spring Branch at a point down stream from the existing dam with the idea of catching the leakage that passed from the upper dam. Leakage soon developed in the reservoir thus formed as a result of passage of water around the ends of the dam through limestone crevices, although this reservoir did retain a limited amount of water, approximately 30 million gallons. Still later a rectangular concrete basin was placed below the lower dam to catch the leakage from this dam. Water was pumped to the city from the concrete basin and the excess of leakage over demand was pumped back into the reservoir above the dam.

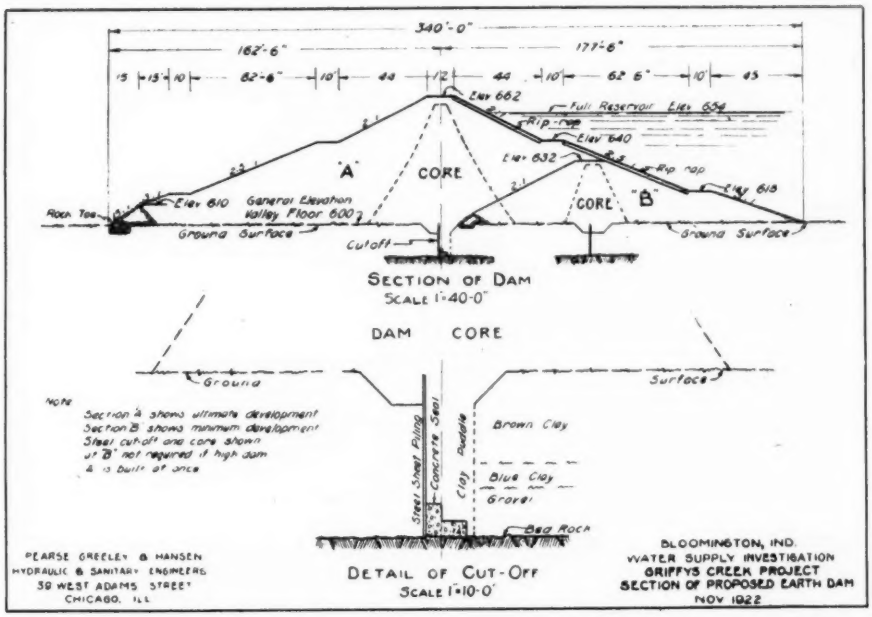


Fig. 1.—Cross Section of Presnet Dam "B" and Cross Section of Enlarged Dam "A." The Existing Dam Is Built of Rolled Earthen Embankment with Selected Material in the Center, While the Enlarged Dam Is to Be Built by Hydraulic Fill Methods

A New Reservoir.—A series of wet years allayed apprehension, but in 1908 there occurred the most severe and prolonged shortage up to that time. This caused the formation of a new reservoir of 50,000,000 gal. capacity known as Weimer's Pond in another nearby and very small drainage area. This reservoir likewise developed some leakage, but not to the same extent as the older reservoir. Again a series of wet years helped out the situation until 1913 when a severe shortage occurred in the late summer.

To settle the question of an improved water supply, the mayor appointed seventeen citizens, all laymen, to report on an improved water supply. Eleven citizens, acting of the advice of a professor of geology at the state university, favored a development in so-called Griffy's Creek to the northward of the city and five favored a dam in a limestone valley to the southward of existing work below and some large springs emerging from limestone, known as Leonard Springs. The presence of the springs, the configuration of the valley and its elevation with reference to the city made this proposition an attractive one to the lay mind and the complicated considerations of tributary drainage area, geological conditions, run-off, dam design and other items that the engineer must take into consideration were not appreciated.

Engineers Retained.—Due primarily to the activities of the same professor of geology who did not believe a vote of a number of lay citizens could determine the best water project, a competent firm of engineers was employed before work was undertaken. The report submitted by these engineers constituted a very comprehensive review of the available sources of supply and recommended definitely development of Griffy's Creek. Griffy's Creek does not lie in the limestone area, but has cut its way through a very impervious clay and sandstone rock locally called knobstone. The configuration of the valley is also favorable to the impoundment of water.

Leonard Springs.—However, the continuous flow from Leonard Springs appealed to the official and popular mind as being more reliable than the intermittent flow in Griffy's Creek and despite engineering advice to the contrary a development was made in the Leonard Spring valley by the construction of an earthen dam 32 ft. high thereby forming a reservoir of about

Fig. 2.—General View of Existing Dam with the Spillway in the Foreground. Through the Other End of the Dam Is a Concrete Conduit Large Enough to Take Flood Waters with Careful Control During Period of Construction of the Large Dam

135,000,000 gal. capacity, together with pumping station and pipe line. This development gave apparently good service for several years but leakage developed around the end of the dam through cavities in the limestone. It further developed that the tributary watershed was not as extensive as appeared on the surface of the ground, namely about six square miles, because the underground drainage through the cavernous limestone did not correspond to the surface drainage. In fact, a very considerable portion of the surface drainage, probably about one third, is diverted by these underground passages into a neighboring watershed.

Other Surveys.—In 1920 a deficiency of water resulted in the employment of a second firm of competent engineers to advise with reference to proper measures for securing an adequate water supply. After reviewing the whole situation, these engineers supported the findings and recommendations of the engineers employed in 1913, with the excepting of minor differences due to additional available information regarding rainfall and run-off.

In 1922 a severe water shortage occurred and this time a firm of engineers of which the author is a member was called in to again review the situation and with instructions to consider in addition another source, namely White River some 14 miles distant. A careful study of all of the factors involved again made it apparent that Griffy's Creek was the only logical and economical development. Nevertheless, opposition to Griffy's Creek was so strong that its actual development was not undertaken until 1924. The general spirit of the criticism is reflected in a remark by a layman to the effect that he knew a frog that lived in Griffy's Creek for four years and it contained so little water the frog had never learned to swim.

New Storage Dam.—A study of Griffy's Creek valley indicated a favorable site for the constructing an earthen dam 2 miles very nearly due north of the City of Bloomington. The tributary drainage area at this point is 8.2 square miles and with storage of 2,850,000,000 gal., sufficient to tide over three dry years, a minimum safe yield of somewhat over 500,000 gal. per square mile per 24 hours can be obtained. This represents for the total drainage area a yield of approximately 4,500,000 gal. per 24 hours, which is sufficient to meet the estimated requirements of the city until about 1960. In addition there are several nearby drainage areas that can be developed in such a manner as to supplement the Griffy's Creek project following the year 1960.

To Be Increased Later.—The City authorities did not favor the full development of Griffy's Creek at the present time and accordingly instead of building a dam 60 ft. high above the valley floor, a dam was built with a total height of 30 ft. and impounding 500,000,000 gal. This height was chosen because it is the largest dam that can be advantageously incorporated as a part of a larger hydraulic fill dam. To have built it lower would have resulted in inadequate storage to meet present and reasonable future demands, even though supplemented by the existing supply. The present development will yield approximately 1,600,000 gal. per 24 hours in a very dry year and 2,500,000 gal. in an ordinary year. The old supply may be counted on for about 1,000,000 gal. per 24 hours in a dry year. The present demands average approximately two million gal. per 24 hours. The accompanying Fig. 1 shows how the smaller dam may be incorporated into the larger hydraulic fill dam.

Pumping Station and Filters.—The city in 1924 did not wish to expend money for recommended purification works in connection with the new development hence a cheap temporary pumping station was built to deliver water through a two mile pipe line to the city. After the new supply was placed in use, however, the water at times was turbid and objectionable to water takers. Accordingly, early in 1927 the city authorized the construction of a permanent pumping station and a filtration plant of 2,000,000 gal. nominal capacity per 24 hours but so designed that it may be readily expanded to a nominal capacity of 8,000,000 gal. daily capacity. This filter plant and pumping station is a substantial and architecturally pleasing structure but involves no especially novel features. The filter plant comprises an aerator, reaction chambers, sedimentation basins, filters and clear water storage. The pumping station has an appropriate number and sizes of pumps for low lift, high lift and filter wash water, all electrically driven. There are also gasoline engine standby units on low lift and high lift services. Additional settings are provided for future pumps.

Method of Financing.—Inasmuch as

Fig. 3.—General View of the New Filter Plant and Pumping Station Nearing Completion

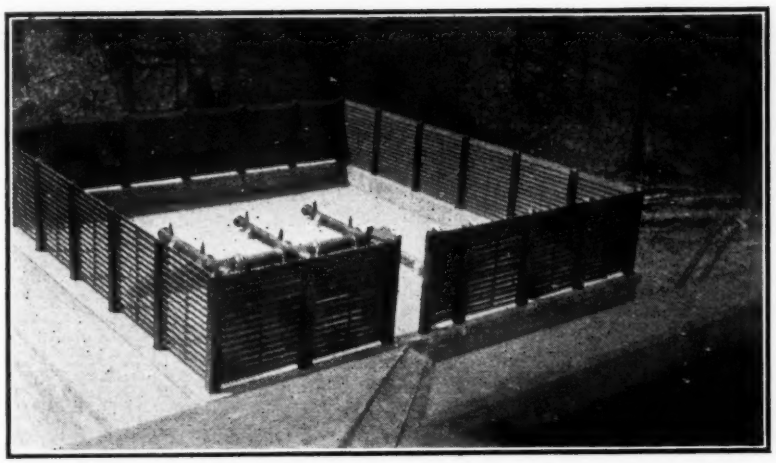


Fig. 4.—View of Aerator Over Sedimentation Basins

the city did not have bonding power sufficient to build the new works, a method of financing applicable to Indiana conditions was adopted. This involves the formation of a water company. The company issues two classes of securities; namely, common stock and preferred stock. The preferred stock under the law may amount to twice the common stock. The city then purchases the common stock with the proceeds of a bond issue which gives the city control of the company. The preferred stock which usually yields 6 per cent is retired serially over a period of years. Thus, the city may finance an undertaking calling for an expenditure of approximately three times the amount of its bonding power.

Old Works Retained.—The present new development in Griffy's Creek is in ordinary years capable of meeting demands but the old works, obtaining water from the several sources previously mentioned, are maintained in readiness to operate in case of large conflagrations or in case of prolonged drought. This arrangement is somewhat inconvenient and does not permit of best operating economy. The next step in water supply development will be construction of a large high service reservoir on the distribution system to avoid keeping the old works in readiness for peak demands and fire emergencies. After that, probably before 1940, the dam must be raised to the full height and the filter plant enlarged. It will then be possible to abandon the old supply entirely.

Experience at Bloomington illustrates the practical difficulties in securing adequate rational and progessive development of public water supplies to reasonably anticipate demands placed upon them. In fact developments are apt to drag years behind demands. This experience also illustrates the time element necessary to bring about improvements in municipal water works after the need for such improvements is realized. In general radical improvements in municipal water supplies require from five to ten years for their realization. The remedy lies in placing municipal water works on a self sustaining basis and providing continuous engineering and economic supervision. How long will it take municipalities to appreciate this?

Board of Engineers to Make Survey of Los Angeles Water System

The Board of Water and Power Commissioners of the City of Los Angeles, through its Vice-President, John R. Richards, has announced the appointment of a board consisting of two engineers and one geologist to make a survey of the whole Los Angeles water supply system. The personnel of the board selected is, as follows:

Louis C. Hill, of Los Angeles, member of the firm of Quinton, Code & Hill, consulting engineers. He has been consulting engineer on many projects on the Pacific Coast, some of the important ones being the Roosevelt Dam, Elephant Butte Dam, Laguna Dam, Gibraltar high arch dam at Fresno, and on the proposed Boulder Canyon dam across the Colorado River.

Robert T. Hill, is a consulting geologist, was formerly Professor of Geology at the University of Texas during 1890-91, was consulting geologist for the U. S. Geological Survey in the investigation of underground waters; was a special Commissioner from the Geological Survey in an investigation of the Martinique eruptions, and is now Professor of Geology at the University of Southern California.

Robert E. McDonnell, is consulting engineer and member of the firm of Burns & McDonnell Engineering Co., with offices in Kansas City and Los Angeles. He has for 30 years specialized in hydraulic problems, their firm having designed water works projects in something over 375 cities, including such municipalities as Springfield, Ill., South Bend, Ind., Lincoln, Neb., Sacramento, Calif.

The appointment of this board has been brought about by the Water and Power Commissioners largely because of the failure of the St. Francis Dam. Their duties will be to make a thorough survey and investigation of the entire water supply system of Los Angeles, including a thorough investigation of the present dams, pipe lines, reservoirs and other work. This special board will consider the feasibility of other methods of storing water than they now practice at Los Angeles, such as storing large volumes of water either in or close to Los Angeles. Their report will cover the feasibility of providing storage for the summer months and locations that would be no menace to nearby communities. The board, began its investigations on May 10.

Municipal Uses of Water at Springfield, Mass.

The following tabulation from the 54th annual report (1927) of the Water Commissioners of Springfield, Mass., shows the metered and unmetered (estimated) uses in 1927 of filtered water from the Little River supply system by the various munipical departments of the city:

Municipal Uses of Water—1927

(Amounts and quantities shown are for mean year periods. Unmetered quantities in gallons are estimated from amounts, on basis of 6⅔ ct. per 1,000 gal., or equivalent of 5 ct. per 100 cu ft.)

Classification	Amounts	Gallons
Schools	$ 8,178.05	122,202,500*
Public Buildings (not schools)	3,304.95	49,192,000*
Public Parks	3,544.95	53,174,200*
Public Playgrounds	377.61	5,664,200*
Building Purposes (Streets and Engineering Department)	336.75	5,051,300†
Sewer Department (flushing)	3,606.08	54,091,200†
Snow Removal (via sewers)	2,684.74	40,271,100‡
Street Washing and Sprinkling	2,681.63	40,224,400†
Drinking Purposes (for animals)	60.00	897,700†
Total of above	$24,774.76	370,768,600
Fire Hydrant Service	60,550.00	§
Total Municipal Use	$85,324.76	

*All metered. †Unmetered. ‡Metered. §Not reducible to quantities. At $25 per hydrant.

The report comments on the above figures as follows:

"The foregoing total amount of $85,-324.76 is a portion of the waterworks accrued earnings of the past year, and as such has to be duly entered in our accounts, but the total amount has to be annually charged off as uncollectible, because payment of same is prohibited by city ordinance."

Central State Water Works Men to Meet in August.—The 1928 convention of the Central States section of the American Water Works Association will be held at the Prichard Hotel, Huntington, W. Va., on Aug. 23 and 24.

Asphalt Grouting for Waterproofing and for Stopping Leakage

How Leakage Was Stopped at Columbia Dam in Tennessee

By GEORGE W. CHRISTIANS

THE Columbia Dam of the Southern Cities Power Co. is located near Columbia, Tenn., on the Duck River. It was built in 1926. It is a gravity section giving about 14 ft. effective head and is founded on dolomite.

The Problem.—The east end of the dam terminates against a nearly vertical rock cliff. The base of the dam is all on rock. The west end of the dam terminates in a sloping bank of aluvial gravel and sandy loam, over the rock. The abutment was carried into the bank some 15 ft. or 20 ft. to apparently watertight formation, but shortly after completion leakage developed at this abutment. In addition wave action and eddies at high water periods eroded the bank just below the dam so that additional reinforcement at this abutment became necessary. A small amount of excavation exposed the outlet of the principal leak and the end of the dam which sloped up from rock on about a 60 degree angle against 4 to 5 ft. of gravel, which, at this point seemed to be tightly filled with clay, with about 15 ft. of firm sandy loam above.

The leakage amounted to between 2 and 3 cu. ft. per second, (see Fig. 2) and came principally from an opening in the rock just under the end of the dam. Other smaller leakage appeared at the head of the trench exposing the main leak and at various points along the bank 50 ft. to 75 ft. down stream. Later work indicated that the smaller distributed leakage was from an entirely separate passage through the rock under the dam.

As a preliminary to reinforcing this end of the dam a contract was entered into with the American Asphalt Grouting Co., of Chattanooga, Tenn. (licensed under Christians asphalt grouting patents) to stop the leakage and waterproof the ground above the dam so that excavation work might be carried on without interference from head water leakage.

Stopping the Leakage.—Holes Nos. 1 and 2 were drilled down about 6 ft. up stream from the dam about opposite the leak outlet, as shown on the sketch. These encountered porous gravel over the rock but did not make connection with the passages in the rock. They were grouted with asphalt however. About 18 bbl. of asphalt were pumped into these holes, which impregnated the porous gravel and reduced the leakage somewhat but did not effect closure of

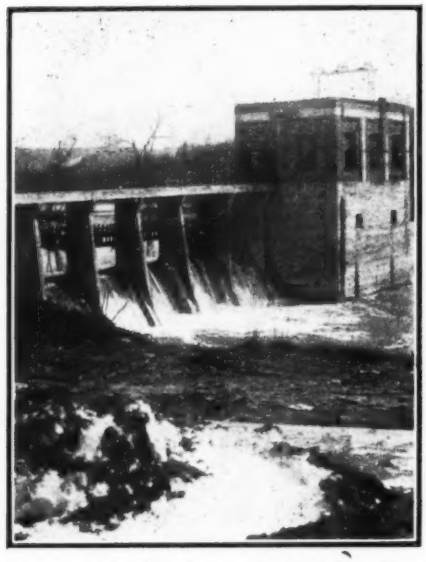
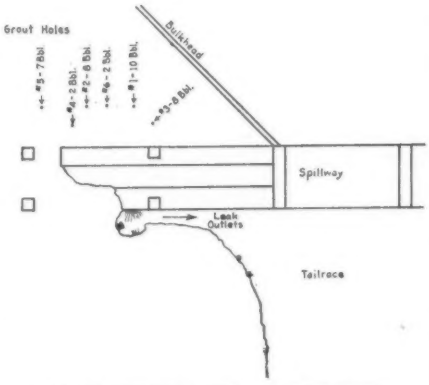


Fig. 1.—Columbia Dam of Southern Cities Power Co.



Sketch Showing Location of Drill Holes

the passages in the rock whose inlets were apparently considerably up stream. Other holes were put down. The one designated as No. 3 on the sketch made direct connection with passage of the main leak about 5 ft. down in the rock.

Pumping of asphalt was started in this hole at about eight o-clock one morning. Almost immediately the leak outlet bcame very turbid as mud and sediment was washed out by the increased velocity of the water as the asphalt expanded in the passage. About 30 minutes after starting a neck of asphalt appeared. By slowing down the pump and reducing the temperature the outflow of asphalt was controlled to a negligible quantity while the mass was expanded in the passage. In another hour the leakage from the main outlet was practically stopped. Pumping was continued, however, until noon in order to get a good body of asphalt on all sides of the drilled hole. Operations were then shut down for about two hours to permit the asphalt in place to cool and harden. Then the injection pipe was reheated by the electrical element and a pressure of 200 lb. per square inch applied which had the effect of slowly squeezing the cold asphalt tightly against the rock surfaces in all directions so that 100 per cent closure was effected and no leakage whatever could be detected at the main outlet.

This work had no effect on the other leakage, part of which is shown in Fig. 3, but the following day the other passage was closed in a similar manner by pumping into hole No. 5 (see sketch) which made connection with the other passage in another stratification of the rock about 3 ft. higher in elevation.

Several other holes were put down and some of them grouted to make sure of the water tightness of the gravel so that excavation work might not be interfered with. In all about 37 drums of asphalt were used.

Fig. 4 shows the final conditions but with tail water too high to uncover the outlets.

The Principle of Asphalt Grouting.—The principle of operation of asphalt grouting is to pump a material which is fluid when hot, but not absolutely solid when cooled, such as asphalt, down through electrically heated pipes placed in drilled holes, into the crevices in the rock below. When the asphalt leaves the pipe and touches the water it congeals on the outside into a nearly solid condition which is tough enough to resist the pressure of a moving stream of water, but soft enough so that a moderate pressure from the pipe line can expand it from the inside until it fills and effectually seals the crevice against the flow of water.

It is necessary that the material be an insulator of electricity, a poor conductor of heat, insoluble in water, that it will melt at a moderate temperature, and that when cold it be stiff enough to resist whatever water pressure may be encountered but ductile enough so that a heavier pressure from the pipe line will slowly expand it into the crevices so as to make intimate contact with the rock surfaces. Asphalt fulfills all these requirments very well. In addition, when so placed it is as permanent as the rock itself.

Fig. 2.—The Leakage Amounted to Between 2 and 3 Cu. Ft. per Second

Fig. 3.—View Showing Part of Leakage

Fig. 4.—Final Condition

When the asphalt emerges from the pipe into water it tends to assume a globular form. (Fig. 5.) The surface in contact with the water is congealed and forms a tough sac enclosing the hot fluid interior. As long as the flow from the pipe is continuous a fluid connection is maintained and the sac can be expanded until externally restricted. The strength or thickness of the enclosing membrane is controlled by the rate of expansion of the sac. When restricted the asphalt assumes the form of the cavity and expands in the direction of least restriction.

The heat conductivity of asphalt is such that as long as the flow is continuous a fluid is easily maintained through the interior of the mass from the pipe line to the point of expansion, and the asphalt can be moved to considerable distance. In a 2 or 3 in. horizontal seam it is not at all difficult to force the asphalt several hundred feet if desirable (usually it is not). This is readily controlled by stopping the flow long enough for the fluid passage to solidify, then when operations are resumed the asphalt must start out again at the pipe until the crevice is completely filled adjacent to the pipe.

When the cavity adjacent to the pipe is filled pressure is applied and maintained for a considerable time, although there may be relatively little flow of asphalt. This pressure has the effect of slowly moving the cold material until the hot material breaks through to some nearby unfilled space, or, if this is not possible, it squeezes the asphalt tightly to the rock surfaces no matter how irregular, so as to preclude absolutely the passage of water. The distance from the pipe at which this pressure on the cold asphalt is effective depends upon the size and nature of the crevice, the grade of asphalt used, the amount of pressure applied, and the duration of the application of the pressure. Usually it is desirable to start with a soft grade to fill the small spaces, then if necessary this can be backed up by a plug of hard material which as it is introduced squeezes the soft material into the small branching crevices.

In operation the pipe may be surrounded by water and thereby kept so cool that the asphalt touching it is solid, but the heat conductivity of the asphalt is such that a fluid passage is easily maintained next to the hot wire in the center so that the pressure is free to emerge at any of several cavities which may exist throughout the extent of the perforations, and when back pressure is built up a definite assurance is obtained that all crevices encountered by this hole are filled. It is in effect a test at a definite factor of safety not only of the watertightness but also of the strength of the foundation material. The heating device not only keeps the asphalt in a fluid condition until it reaches the cavities to be filled but also makes it possible to resume operations after a shutdown merely by applying the heat for a few minutes before starting the pump.

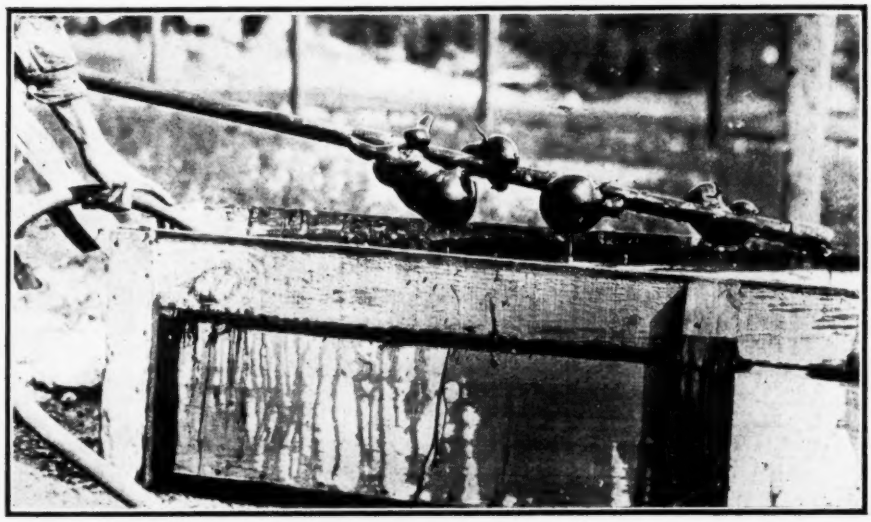


Fig. 5.—Asphalt Emerging from Electrically Heated Pipe Line

New Water Works and Filtration Plant at Laredo, Texas

General Features of the System Described by Designer

By R. T. REILLY
With Alvord, Burdick & Howson, Consulting Engineers, Chicago

THE City of Laredo, Texas, is located on the Rio Grande approximately 140 miles south of San Antonio. The present population is estimated at about 30,000. The present average daily and maximum daily pumpages are approximately 3 million gallons and 4.75 million gallons respectively.

Laredo has been supplied with filtered Rio Grande water for many years from an old water works station located near the business district on a site closely adjoining the International Bridge. This plant is so located that it could be protected against floods only with considerable difficulty and the pollution incident to the growth of the City tended to increase the difficulty of adequately treating the water.

To overcome these difficulties it was decided to purchase a new site located up-stream from the source of pollution and on high ground which lies above the highest known flood waters on the Rio Grande.

Units Included.—The new plant includes an intake, low lift pump house, mixing and coagulating basins, filters, clear well and a high lift pumping station. It is connected with the city distribution system through approximately 2 miles of 20 in. cast iron main.

The intake shown in Fig. 1 is constructed of reinforced concrete and is located at the edge of the river. It is equipped with gates and screens in duplicate so that a continuous supply may be drawn while cleaning one part of the intake. The use of this type of intake was based upon the uncertainty of the permanency of a pipe intake extending into the river, the bed of which is subject to excessive erosive action during floods.

Low Lift Pumps.—The intake is connected to the low lift pumps by 2-24 in. C. I. pipes, each approximately 100 ft. long. The low lift pumping installation shown in Fig. 2 consists of two electrically driven pumps, each capable of pumping 6 million gallons per day against 70 ft. head. The water is delivered from these pumps to the aerators, mixing chamber and coagulating basins.

The aerator shown in Fig. 3 consists of 24 nozzles spaced 5 ft. apart and located on top of the mixing tank. Under certain conditions which frequently prevail in the Rio Grande, the water will be passed through these nozzles for the purpose of eliminating tastes and odors due to the presence of CO_2 and organic matter present at times in large amounts.

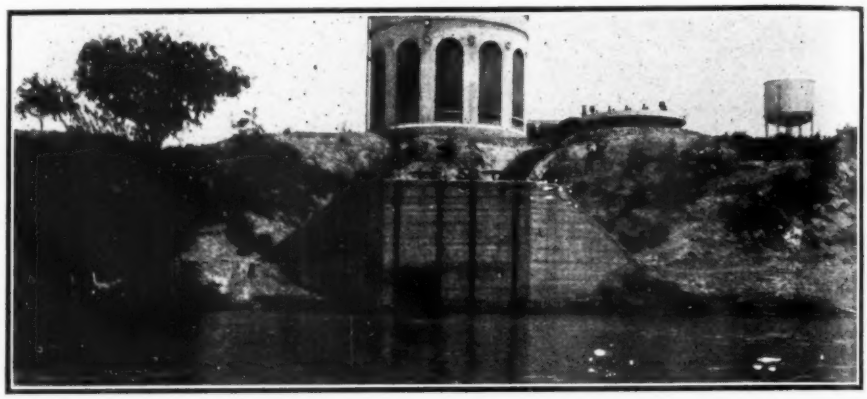


Fig. 1.—View of Intake From River

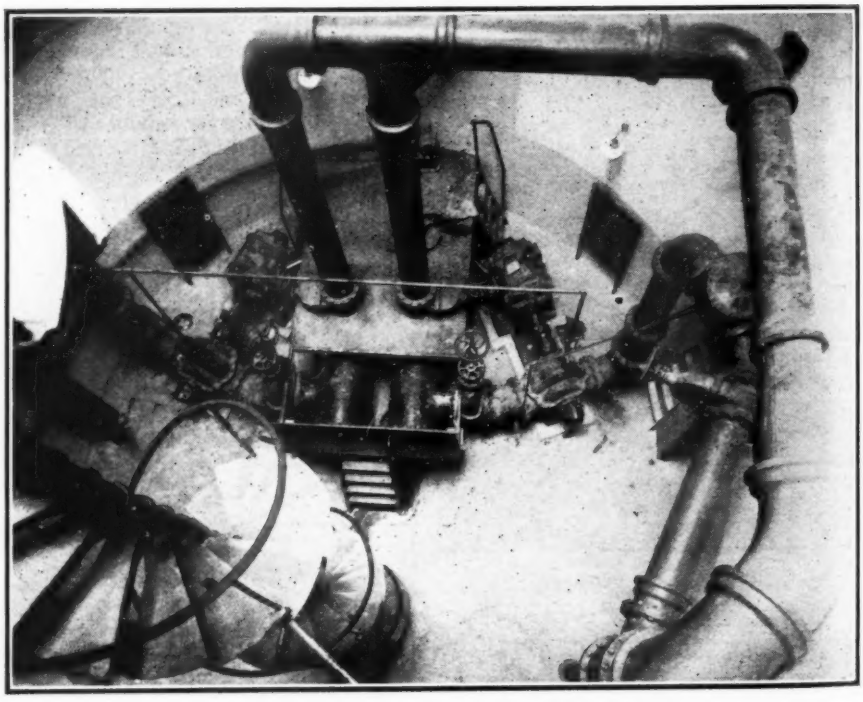


Fig. 2.—Interior View of Low Lift Pump Room

Chemical Treatment.—Normally, it is expected that the water will first be delivered into the mixing tank for treatment with coagulants. Facilities have been provided for the feeding of either or both lime and alum coagulants, either simultaneously at the same point or at different points in order to provide that flexibility of treatment which experience at Laredo indicates as desirable in order to obtain the best results in coagulating and filtering the water under the varying conditions occurring in the Rio Grande.

The mixing tank is circular in plan and the entrance and exit piping has been so arranged that the mixing of the water with the chemicals will be accomplished by the peripheral velocity without mechanical agitation.

Settling Basins.—From the mixing tank, the water passes to two circular

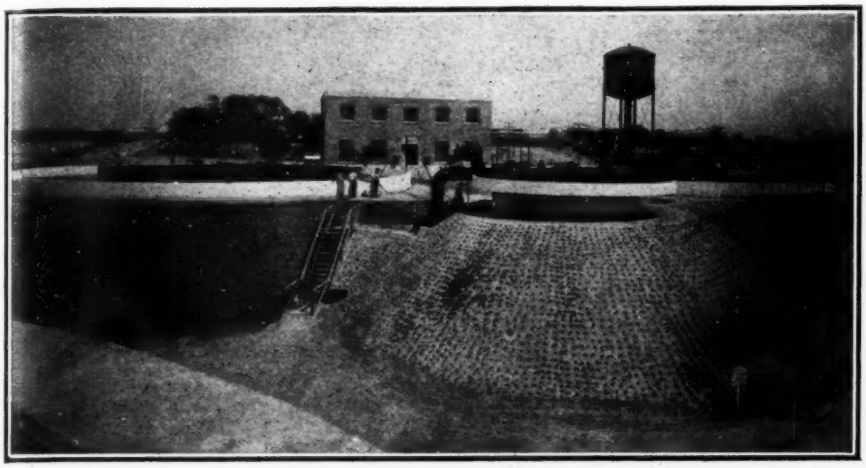


Fig. 3.—General View of the Plant Showing Mixing Tank and Settling Basin in the Foreground

settling basins shown on Fig. 3. The inlet and outlet distributing facilities of these basins consist of slotted wood baffles made up in sections as to permit by test and observation of accomplishing their installation in such manner as to produce uniform velocities across the basins and thereby result in the most efficient operation thereof.

Two settling basins have been provided so that one basin will be available for use while the other is being cleaned. The total capacity of these basins is approximately 1,500,000 gal., and the total detention period thus provided for is approximately 6 hours, based on the rated capacity of the plant.

The valve and piping arrangement between the mixing tank and settling basins is such that the water from the low lift pumps may be delivered directly to the aerator or mixing tank or to either or both of the two settling basins.

Filter Units.—From the settling basins the water flows to four gravity mechanical filters shown in Fig. 4 having a total capacity of 6 million gallons in 24 hours.

The filters are equipped with 16 in. C. I. manifold headers and 2 in. C. I. laterals spaced 6 in. on centers perforated on the bottom with 5/16 in. holes on 6 in. centers. The bottom of the filter is covered with a 30 in. layer of gravel grading from sizes retained on 2½ in. to ⅛ in. screens. A 30 in. layer of filter sand having an effective size of .49 mm. and a uniformity coefficient of 1.36 is placed on top of the gravel. The lip of the wash water troughs is located 24 in. above the top of the sand and the troughs are so spaced that the travel of the water does not exceed 3 ft. in a horizontal direction before entering the wash trough.

Wash Water.—The wash water troughs, underdrainage system and connecting piping have been designed to permit washing the filters at a rate not less than 15 gal. per minute per square foot of filtering area. The operations of filtering, washing and rewashing each filter unit are controlled by hydraulically operated valves manipulated by control handles assembled on operating tables conveniently located on the operating floor.

A clear well reservoir containing 250,000 gal. is located underneath the filters. This reservoir serves as a suction sump for the high lift pumps, the variations in demand being met by elevated storage of 2,000,000 gal. located on high ground on the opposite side of the business district from the plant.

Pumping Station.—The new pumping station shown in Figs. 3 and 4 closely adjoins the filters and clear well. This building, a substantial brick structure is being equipped with two electrically driven pumps shown in Fig. 5, each capable of pumping 6 million gallons per 24 hours against 250 ft. head. Space is provided for two additional pumps. These pumps deliver their water to a 20 inch cast iron main about two miles in length which connects to the city distribution system in the northern part of the business district. Feeder mains also connect to the large reservoir. The operation of the high lift pumps is governed by a water level gage which indicates and records the water levels in the elevated reservoir.

The pumping station is a two story structure. The upper story is used for the storage of alum and lime for treating the water. It also contains an office and a laboratory in which daily analyses of the water will be made. Alum and lime will be fed by automatic dry feed machines capable of feeding the water at a pre-determined rate in conformity with the rate of pumpage indicated by the station master meter.

The grounds immediately surrounding the pumping station are built up with materials excavated from the reservoirs so as to give access to the pumping station and grounds even in periods of very high water. All door sills, window openings and the tops of all basins are located above the high water mark.

The Owners.—The Central Power and Light Company own and operate the Water Works. E. B. Neiswanger is President, S. M. Udden is General Superintendent, W. D. Alexander is Local Manager at Laredo and H. J. Seaman is Superintendent of the Water Department.

The McKenzie Construction Company of San Antonio were the contractors on the structures and installation of yard and supply piping. The International Filter Company of Chicago furnished and installed the filter and purification equipment and station piping.

The pumping station and filter plant were designed by Alvord, Burdick &

Fig. 4.—General View of the Plant, Showing the Filter Units and Wash Water Tank

Howson, Engineers, Chicago. This firm is represented upon the construction by H. T. Rudgal, resident engineer.

Getting Good Results from a Filter

Practical Suggestions Given at 2nd Annual Conference Maryland Water and Sewerage Association

By A. L. GOSNELL
Filtration Plant Operator, Laurel, Md.

The object of a filter being to convert a turbid water into a clear water, it is well to have the entire area of that filter working. Any obstruction, whether it be air binding, mud balls or gas, cuts off that portion of the filter and throws an overload on the remaining portion. We will assume that the filter is washed at proper intervals. The writer prefers to wash a filter three or four hours before it needs it, rather than wait until the filter begins to slow down. This is a matter that is entirely up to the operator. The rate-of-flow controller is a very good indicator as to what is going on in your filter. If your filter is in good condition the controller arm will raise and control the rate of flow. When the petcock in the effluent pipe is opened water should flow from that pipe. If on opening the petcock no water comes, it indicates an obstruction in the filter and that there is not enough water passing through to entrain the controller. The filter may need washing or it may be air bound. If air bound close effluent valve, open wash valve four or five turns and force air out by turning on high pressure stream through wash pump.

Scraping Filters.—There is one job that should receive special attention and that is scraping filters. There is a portion of mud that is too heavy to be raised to the top of the wash troughs by washing filters, consequently it settles down and remains on your filter. This continues day after day until it interferes very materially with your work. This mud must be scraped off and taken out. I will not attempt to say how often a filter should be scraped. Some say once a month but it is better to scrape it once every two months than to say once a month and not scrape it at all. If there is any one thing that will put new life in a filter and put it in such shape that it is a pleasure to operate it, scraping will do it.

Washing Filters.—Washing filters deserves our careful consideration. To wash a filter, lower the water to about 3 in. from the sand, close effluent valve, open waste valve, open wash valve four or five turns, turn on wash pump, and open wash valve gradually until wide open; let remain open as long as necessary. Close gradually but leave open four or five turns. Open high pressure 1 in. line to top of wash pump and cut out pump. Close waste valve. In our plant we have a 1 in. high pressure line leading into top of wash pump. This line is opened just before the pump is shut down and remains open after pump is stopped. This permits a stream of approximately 30 gal. per minute to pass up through the filter from the bottom and distribute the flock evenly over the surface of the filter. This 1 in. line should remain open until the filter is nearly filled with water from the coagulating basin, but must be closed preferably five or ten minutes before putting filter in service. The wash valve must be closed immediately after one inch line is shut off. This method of procedure has been used in place of rewash for the past two years at our plant with good results.

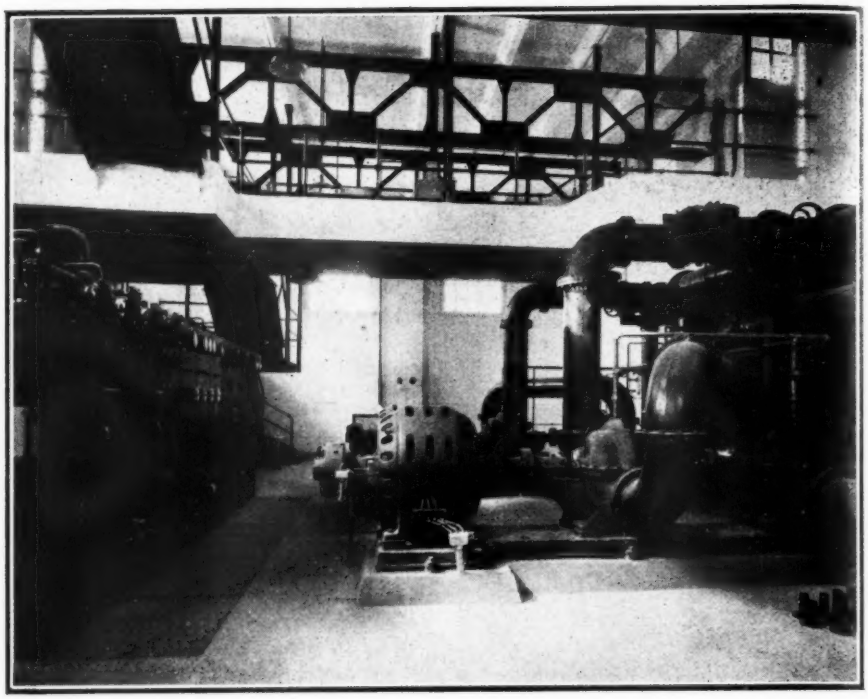


Fig. 5.—Interior View of High Lift Pump Room

It is the general opinion that a filter does not do its best until it has been in service one hour. The object of this method is to get the advantage of that one hour. After washing a filter in the usual manner, if the rewash is used before a good blanket has been formed over the surface of the filter, the movement of the water is downward at approximately 5 in. to the minute, carrying a portion of mud below the surface, while the other method works in the opposite direction. By opening a high pressure stream through the wash valve the movement of the water is upward at about 1½ in. to the minute. This is not enough to disturb the sand bed and does aid materially in spreading a flock evenly over the filter. In taking a sample of water to check up on the efficiency of a filter that has just been washed, the filter should be in service about 15 minutes before sample is taken. This will take out the water that was left there by the wash pump and give you the first water passing through from the coagulating basin. Samples taken at our plant 15 minutes after washing show a very high rate of efficiency.

Cost of Pumping Water at Columbus, O.

The total pumpage to the city of Columbus, O., for 1927 was 10,210 million gallons, which is equivalent to an average daily pumpage of 28,000,000 gal. The maximum daily pumpage was 39,500,000 gal. and the minimum daily pumpage was 22,700,000 gal. The maximum rate of pumping occurred on July 13, 1927 and was 59.5 million gallons per 24 hours, which is 210 per cent of the average rate for the year and 150 per cent of the average rate for the day on which it occurred. The maximum daily pumpage was 140 per cent of the average daily pumpage and the minimum daily pumpage was 80 per cent of the average.

According to the 1927 report of C. B. Hoover, Superintendent of Water Works the cost of pumping water to the city, including raw water pumping, direct service pumping and booster station pumping, for the past seven years and on the basis of the total gallons delivered to the city was as follows:

Cost of Pumping Per 1,000,000 Gal.

Year	Labor	Coal	Electric Power	Other Costs	Total
1921	$8.31	$6.44	$0.83	$1.12	$16.70
1922	8.06	7.98	1.10	1.29	18.43
1923	7.45	4.78	1.35	1.28	14.86
1924	7.60	4.50	1.33	1.12	14.55
1925	7.50	4.20	1.15	1.45	14.30
1926	7.23	4.24	1.14	1.71	14.22
1927	7.52	4.36	1.08	1.06	14.02
Means	7.57	5.63	1.11	1.16	15.48

Recreational Use of Water Sheds of Public Water Supplies

A Review of the Situation in the State of Pennsylvania

By W. L. STEVENSON and H. E. MOSES

Respectively, Chief Engineer and Assistant Chief Engineer, Pennsylvania Department of Health

A feature of water work management which is bringing increased concern to water works officials and to health authorities is the recreational use of the catchment areas of streams used as sources of public water supplies. An interesting discussion of this situation in Pennsylvania was given in the paper by Messrs. Stevenson and Moses, presented at the last annual convention of the Pennsylvania Water Works Association.

RECREATIONAL use, generally construed to mean the temporary use of human beings on a watershed who may indulge in bathing, boating or fishing in a stream, lake or pond, and camping, hunting, picknicking or hiking over the area.

Until recently this question was not so pressing because of the inaccessibility of many watersheds, but with the development of the automobile and the improvement of the highways, the state's mountain and lake regions have become accessible to our citizens and thousands of people, who were formerly denied this pleasure, are now enabled to visit these sections, thus increasing the potential menace to the public water supplies derived therefrom.

Public Demand for Use of Catchment Area.—In Pennsylvania, a water company is required, under its charter oblitigation, to furnish the public "pure" water, which in general means a supply safe and suitable for domestic use, and it may be said that the water companies in Pennsylvania are zealously and earnestly striving to meet this duty. However, they are confronted on the one hand by their obligation to furnish a safe water supply, coupled with the insistence of the state health authorities that this be done, and on the other hand by the desire and even the insistence of the public to use the catchment areas and, in some instances, even the reservoirs for recreation.

The most pressing demand perhaps is the right to camp and hunt on the watershed and to fish, and, to a lesser degree, to bathe in the streams used as sources of public water supply.

The Present Day Tendency.—That the pressure in this direction is real is evidenced by the requests received by the state authorities for permission to use streams and reservoirs of public water supplies for fishing purposes; by the rapidly increasing number of camps and summer colonies or watersheds, whose occupants insist on the unrestricted use of the streams for their recreational pleasures; and by the passage of legislation in at least one state throwing open virtually all streams for bathing, seemingly to controvert regulations passed by the state health department as a protective measure.

In the matter of legislation, it is interesting to quote from an act passed by the New Jersey legislature during the current year, vetoed by the governor, repassed over his veto, and so now a law. This statute reads, "it shall be lawful to bathe or swim in any of the fresh waters of this state, provided that in so doing no trespass be committed. This act shall take effect immediately." This probably represents an extreme measure, but is cited to show a present day tendency.

Riparian Rights.—Akin to this phase of the question is the matter of riparian ownership of land abutting on water used as a source of public water supply. It is understood that such riparian owner has an inherent right to the reasonable use of the stream, including bathing, for himself, his family, servants, and invited guests. This probably holds so long as the riparian owner does not commercialize this use of the stream.

The right to fish in domestic water supply streams and reservoirs is being insisted upon. The fishermen's organizations seek this privilege from the water companies and from the state authorities. Some water companies are unalterably opposed to such practice, while others allow it, in some instances, granting permits and—so rumor has it—actually charging a permit fee. The same Dame Rumor intimates that there are cases where the water company permits only its own officers and friends to fish on its property, thereby incurring the ill-will of the fishing public not similarly blessed.

The above merely serves to emphasize the difficulty in adopting and enforcing measures relating to the recreational use of watersheds for the protection of public water supplies, which shall at once be reasonable and practicable and at the same time not serve to arouse such antagonism on the part of those desirous of using the watersheds for such purposes as to result in a popular clamor which might bring about the adoption of such legislation as previously cited.

Classes of Watersheds.—In order to bring the subject within reasonable bounds, it might be well to attempt to set up certain classes of watersheds and determine if possible the effect on public water supplies of their use for recreational purposes.

The uninhabited watershed today is a rarity, especially in Pennsylvania, and it is coming more and more to be believed that all surface supplies are potentially dangerous and cannot be considered safe without protective features.

There are certain streams in the state used as sources of public water supplies, principally the rivers and major tributaries, draining developed areas, having a relatively large volume of flow, used with a maximum of purification measures and where filtration of the water is the almost universal practice, wherein recreational use might be permitted with reasonable safety.

The other extreme is a class of streams, usually the smaller ones, draining an undeveloped area, having a relatively small volume of flow, with a minimum of purification measures and no long time storage. Here it is somewhat obvious that the safety of these supplies depends upon absolute prohibition of the use of the watershed for recreational purposes.

In between these two, there are many streams not falling into either of the aforementioned classifications, where possibly recreation could be permitted under certain restrictions.

It is quite evident, therefore, that there can be no blanket regulation on this subject having state-wide application. Local conditions will have an effect upon each case.

It would, however, be quite advantageous if certain principles could be erected and regulations adopted for the guidance of the water works officials and of the state authorities.

Pennsylvania Zoning Regulations.—An attempt has been made in this direction in the zoning regulations adopted by agreement between the Department of Health and the Depart-

ment of Forests and Waters of Pennsylvania.

Three zones are established on watersheds of streams draining State Forestry Lands and used as sources of public water supplies. Within the first mile by stream above the intake, camping is not permitted on the watershed except where existing leases are held, which at their expiration will not be renewed. Within the next mile leases are granted under stringent regulations as to sanitary conditions, all aiming to protect the stream. The third zone constitutes the remainder of the watershed above Zone No. 2, and camping here is also permitted, the regulations being slightly less stringent, but sufficient to give reasonable protection to the stream.

The permits are revocable for cause, one being disregard of sanitary regulations, and as a rule the camps are inspected annually. Unfortunately, from the standpoint of such control, the land owned by the state forms a comparatively small portion of the area involved in this problem.

There is also a printed statement on the license issued in Pennsylvania to the fishermen warning against stream pollution.

Sanitary Patrol of Watersheds.—In water works permits the Pennsylvania Department of Health has attempted to establish practices whose purpose is the protection of public water supplies. In practically every permit issued, approving sources of public water supply obtained from streams where the watershed is of a reasonable size and not one of the major streams, it is required that a sanitary patrol of the watershed shall be made above the intake at stated intervals and a report thereon filed with the department, particular emphasis being laid on the necessity of reporting promptly the existence of any pollution or cases of infectious or contagious diseases.

In one or two permits the water company has been charged to take such steps within the law that will prohibit the use of the proposed reservoir for recreational purposes, such as fishing, boating and swimming and also prevent trespassing upon the margins of the reservoir owned and controlled by the water company.

Another protective measure, whose effect can hardly be ascertained, is the posting of signs on a watershed calling the public's attention to the fact that the drainage area is used for public water supply purposes and warning against any act which will jeopardize the purity of the supply.

In a sense, however, these efforts might be considered as palliatives, yet, valuable as they may be, not solving the serious problem confronting the water company.

Two General Principles.—In a further attempt to narrow the subject to a practical basis, two general principles may be applied to a watershed to ascertain if some measuring stick can be adopted which will have more or less general application throughout the state, and in this connection it might be well to consider the "recreational use" of a watershed as comprising bathing, fishing, hunting and camping:

First, where the reservoir does not have a nominal long time storage and where ways and means have not been installed or are not being operated to safeguard the sanitary quality of the water delivered to the consumers and where the catchment area is relatively uninhabited and untraversed by much traveled highways, the recreational use of the catchment area does constitute a menace to the public health.

Second, where the nominal retention of the water in the reservoir can be construed as adequate long time storage and where ways and means have been installed and are being operated to safeguard the sanitary quality of the water delivered to the consumers and the catchment area is inhabited or traversed by extensively used highways, the recreational use of such catchment area does not constitute a menace to the public health because the persons so using these areas does not add an undue burden upon the self-purification to be obtained from the assured long-time storage and from the artificial ways and means which have been installed and are being operated to safeguard the sanitary quality of the water delivered to the consumers and hence, in such cases, consideration might be given to permit the public a restricted use of the watershed area, exclusive of the reservoir and the margins thereof.

Remedies Summarized.—What then are the remedies for this situation? A few which have been suggested are summarized briefly below, in each instance pointing out the advantages and disadvantages.

(1) Ownership of Watershed.

This is basically an economic problem and must be determined by the respective merits of ownership versus protective measures and purification. Its effectiveness is dependent on the extent of watershed control and grows less practical as the size of the watershed increases. Where it owns the watershed, the water company can invoke trespass laws. Certainly the reservoir site and the shore margins should be owned or definitely under the control of the water company.

(2) Zoning.

This practice is not applicable except on company controlled or state owned land. Elsewhere private ownership prevents.

(3) Placarding.

Practical on small areas. Effectiveness uncertain, but probably beneficial.

(4) Sanitary Control

Within rather circumscribed limits, can be administered under existing laws, health regulations and with available personnel.

(5) Education of Public.

Undoubtedly of value, but must be persistent and continued. Will not control the careless or vicious.

(6) Legislation.

Must be reasonable, capable of enforcement, with sufficient elasticity to be applicable to the varying conditions encountered. If too drastic, it is difficult of enforcement and arouses militant opposition, possibly defeating its purpose, which may be entirely proper.

Possibly a judicious combination of these various measures may help in solving this problem. As it is of statewide interest, it is obvious that a central agency is needed to administer the provisions of law or regulation governing it. Such an agency is found in the State Health Department, whose province as relating to public water supplies is their sanitary quality.

A safe and suitable water supply is a vital factor in the health and prosperity of the public. It is the duty of a water company or of a municipally owned water works to furnish its consumers with such a supply. Persons residing upon or visiting the watersheds of streams have certain rights and privilages, but they also have an obligation to so conduct themselves that their actions will not prejudicially affect the purity of such supplies. To the end of reconciling these varied obligations and interests, it might be found expedient and practical to adopt control regulations based on existing laws or on new laws, if necessary, and founded on the principle that although they must be of general application, yet they must be administered with due consideration of the controlling features of each individual case.

Financial Return from Installation of Water Works.—In his annual report for 1927 Abel Wolman, Chief Engineer of the Bureau of Sanitary Engineering of the State Department of Health of Maryland, points out the large financial return, in addition to improved health conditions, resulting from the installation of adequate water works. At Leonardtown, Md., a community of between 500 and 600 inhabitants, a public water supply was installed a few years ago. A survey and re-assessment of insurance rates was made during the year by the fire underwriters. This survey resulted in an average reduction of 52 per cent in fire insurance rates. Similar gratifying reductions have been obtained in a large number of other areas. With the prospect of such savings in insurance rates, coupled with adequate fire protection and increased comfort to the residents, no convincing reason remains why every small town or community should not provide itself with public water supply facilities.

Refuse Disposal in Great Britain

Observations Made in the Summer of 1926 Given in a Public Health Bulletin of U. S. Public Health Service

By H. W. STREETER
Sanitary Engineer, U. S. Public Health Service

In the course of the visits an opportunity was provided for obtaining considerable information concerning public systems of refuse disposal and for observing a few of these systems in operation. The following notes have been based on information thus secured, supplemented by data kindly supplied by the Ministry of Health and various municipal engineers.

British Meaning of "Refuse."—In Great Britain the term "refuse" usually is understood to include both trade and domestic rubbish and domestic garbage, as rubbish and garbage ordinarily are not separated at their sources in that country, but are handled together. In a few districts, rapidly dimnishing in number, where water-carriage systems of sewage disposal have not been introduced, "mixed" refuse also includes some fecal matter. Under the term "disposal" may be included the collection of refuse, which almost invaribly is undertaken by the disposing agent.

Under the public health act of 1875, the local authorities may, or when required by the Ministry of Health, must undertake, or contract for, the collection and disposal of house refuse. If the local authorities do not wish to meet the cost of new works out of current revenue, they must obtain the sanction of the Ministry of Health for seeking a loan. This procedure is exactly parallel to that which is necessary when any improvements or extensions of public systems of water supply or sewage disposal are to be financed from borrowed capital.

Cost of Refuse Disposal.—According to information supplied by the British Ministry of Health, the total amount of refuse (excluding street sweepings) collected by local authorities in England and Wales is approximately 12,000,000 tons annually, the collection and disposal of such refuse involving a yearly expenditure of about £8,000,000. Based on these figures, the average cost of refuse disposal amounts to two-thirds of a pound sterling, or approximately $3.24, per ton of refuse. The collection of house and trade refuse is almost universally lndertaken by the local authorities, contract labor being confined mostly to certain small rural districts. The amount of dry refuse, though varying according to the local industries, averages about 1.5 pounds per capita daily. On the basis of this and the above figure, the average cost of refuse disposal may be estimated as being approximately 80 cents per capita per year.

The frequency of collection of dry refuse varies with the locality, being sometimes daily but more often weekly. Dry refuse usually is stored in small galvanized-iron tins kept at the rear of the dwellings and collected at such points.

Refuse Disposal Methods.—In Great Britain, there are five recognized methods of refuse disposal, namely:

1. Deposition on land.
2. Pulverization or crushing, with subsequent use as fertilizer.
3. Incineration.
4. Separation.
5. Disposal in tidal waters.

The deposition of refuse on land, commonly termed "tipping," is still widely prevalent but not generally approved, though it is permitted under favorable conditions, with certain precautions prescribed by the Ministry of Health. These precautions require, among others, that refuse dumps shall be formed in shallow layers, that all refuse must be covered with incombustible material, that distribution of debris by the wind must be prevented, and that no refuse shall be deposited in stagnant water.

The pulverization, or crushing, method involves passing the refuse through a disintegrator, in which it is reduced to a coarse powder by means of hammers. During this process the refuse becomes thoroughly mixed and the final product is useful as a fertilizer for heavy soils. The use of pulverized refuse in this manner does not cause the nuisances usually associated with the tipping of crude refuse.

The incineration method is very similar in its main features to the corresponding method followed in other countries. In applying this method, the crude refuse is burned in specially constructed furnaces, usually with a forced draft, until all of the combustible material has been consumed. No auxiliary fuel is ordinarily necessary, owing to the relatively small amounts of wet garbage in the average mixed refuse. Frequently the heat generated by burning the refuse is used for the generation of steam for power purposes. The unburned residue, the volume of which averages about one-third that of the original refuse, consists of scrap metal, which is drawn from the grates by hand; and clinker, which frequently is ground and used as an aggregate for concrete.

The separation process varies in different localities, but in general has for its object the removal of dust and, in some instances, the salvaging of certain valuable materials, such as rags, paper, and metals, from the crude refuse. This method is frequently practiced in conjunction with pulverization or incineration, either of these two processes being applied to the final residue.

The disposal of refuse at sea is practiced by a considerable number of towns along the seacoast. Hopper barges are used, varying in their capacities from 100 to 1,000 tons. The barges, some of which are operated under their own power, discharge the refuse in deep water at points designated by the authorities. The distances of these points from the shore vary from 3 to over 10 miles.

Inceneration Method of Small City.—At Eastbourne a good example of the incineration method of disposal, as applied to a moderately small city was observed, and at Birmingham, a particularly interesting example of the combined separation and incineration method as applied to a large city. At Eastbourne, the refuse is brought to a central incineration plant located immediately adjacent to the low-level sewage pumping station, and is dumped into a series of hoppers, from which it is fed into the combustion chamber of a 6-cell incinerator. The refuse is burned under a forced draft, without the addition of any supplementary fuel, and the heat thus generated is used to operate three water tube boilers supplying steam at a pressure of 100 pounds. The scrap metal is removed by hand from the grates as it accumulates and is disposed of seperately. The residual clinker is ground and utilized as an aggregate for concrete curbstones and slabs, which are used in connection with the municipal paving and housing schemes. The steam generated by incineration of the refuse is converted into electric power which is utilized for pumping all of the low-level sewage and storm water to the points from which it is discharged into the sea.

From data* kindly furnished by Maj. Leslie Roseveare, borough engineer of Eastbourne, the amount of refuse burned during the year 1922 was 1.55 tons per house, or 0.25 ton per capita. The costs of collection and of incineration, including interest and sinking fund charges, were $2.80 and $1.58 per ton, or $0.70 and $0.39 per capita, respectively.

*Notes on Eastbourne. By Leslie Roseveare, borough engineer. Prepared for a meeting of the Institution of Municipal and County Engineers at Eastbourne, Sept. 22, 1923.

Disposal at Birmingham.—The city of Birmingham, which in 1926 had an estimated population of 961,222, affords an extremely interesting example of the possibilities of the separation method of refuse disposal when carried out efficiently and on a large scale. The system of refuse collection followed at Birmingham also has some very interesting features.

For purposes of refuse collection, the city is divided into eight districts. The collection service in each district is organized according to a series of routes, so that each house visited has its definite place on the working list. If the collectors are unable to obtain access to a particular place a card is left informing the owner of the day of the next collection. At the end of each day the collector reports the last address visited on his route, and a progress chart is maintained showing the number of houses visited. With the aid of this chart, which also shows the average weight of refuse per load, the district inspector is able to determine exactly where the collection is in arrears, to allocate spare collection service accordingly, and to discover any slackness on the part of individual collectors.

Up to the year 1918, the work of refuse collection was carried out entirely by horse-drawn vehicles. The "continuous" system of collection, above described, could not be operated economically with horses, and since 1918 more than 70 electric vehicles have been put into service, their capacity varying from 7 to 13 cubic yards. The salvage department still employs over 100 horses.

The disposal of the refuse is carried out at 10 plants, but it is planned to reduce to 8, or one plant for each collection district. Three of the larger and more modern plants, designated, respectively, as the Montague Street, Brookvale Road, and Tyseley plants, were visited and the details of their operation observed. The Brookvale Road and Tyseley plants are regarded as the best-equipped works of their type in Great Britain. In general, the process of disposal used at the three plants is as follows:

After being weighed in the collection trench the refuse is hauled by the trucks up an inclined roadway and delivered to the plant on an elevated platform, where it is dumped into large hoppers. From here the refuse is taken by a series of automatic feeders and delivered to cylindrical screens, which remove the fine dust and deposit it in hoppers underneath the screens, whence it is removed by a pneumatic conveyor system of barges, which transport it to a prepared "tip" or other place of disposal. After leaving the screens the refuse is passed through a cylinder having a magnetized section, which automatically removes all metal pieces and deposits them in a hopper, whence they are conveyed automatically to a metal pressing and baling plant. The tailings are carried to the incinerators by belts so arranged that while en route miscellaneous articles of value are removed from the belt by hand, and waste paper is removed automatically by a suction apparatus and delivered to a baling plant. The incinerators are operated without the use of any auxiliary fuel.

By-Products.—Among the by-products of this process which have a definite commercial value are: (*a*) Rags, (*b*) paper, (*c*) scrap metal, (*d*) tin, (*e*) clinker, and (*f*) steam power. The rags are obtained by hand picking from the conveyor belt and are sterlized, laundered, and sold for various purposes. The paper and scrap metals are baled and sold to the paper and iron industries, respectively. The tin, which is obtained by a special process whereby waste metal cans are detinned, is recovered only when market conditions are favorable for the recovery of this metal. The clinker, which is a product of the final incineration process, is ground, screened into various sizes, and used as an aggregate for concrete. The steam power generated by the incineration of the tailings from the separation process is converted into electric power, which is utilized for operating the disposal plant and for charging the storage batteries of the electric refuse collection trucks. It is understood that all of the power required for the collection of the refuse by these trucks is supplied by the incineration plant.

In the operation of these various recovery processes, a separate cost and profit account is maintained for each process. If a particular salvaging operation shows a financial loss, it may be discontinued temporarily until favorable market conditions for the sale of the material thus salvaged become restored. At the time of the visit to Birmingham, the recovery of tin from the waste metal cans had been discontinued for this reason.

Organic Waste Utilization Plant—At the Montague Street works, in addition to the refuse disposal system above described, an organic waste utilization plant is engaged in the conversion of the city's animal and fish waste into feeding meals and fertilizers. This process, which is operated through an ingenious system of closed digestors and grinders, is conducted in such a manner that disagreeable odors are reduced to a minimum and appear to be confined entirely to one section of the plant. The products have a very high commercial value. According to figures supplied by the salvage department, about 4,000 tons of animal and fish waste are disposed of annually in this way, with a recovery of 1,000 tons of feeding meals and fertilizers and 80 tons of fats. The annual turnover of the plant is approximately £10,000, and about one-half of the product is exported. The entire operation of the plant is maintained under detailed laboratory control.

During the year ended in March, 1926, 260,832 long tons of refuse were collected and disposed of by the salvage department. Of this amount, about 62 per cent by weight was incinerated; 26 per cent consisted of screenings; 8 per cent represented cesspool contents taken to farms and sewers; and the remaining 3 per cent, material treated at the organic waste recovery plant and salvaged as scrap metal and sundries. The gross expenditure for collection and disposal of the refuse was £254,375 and the gross income, £53,058, or about 21 per cent of the gross expenditure. The net cost, represented by the difference between these two figures, was £0.77 ($3.75) per long ton of refuse, or £0.21 (approximately $1) per capita, as based on an estimated population of 961,222 for the year 1926. The output of refuse per capita for the year was 0.27 (long) ton.

Costs of Disposal.—The operations of the Brookvale Road works, when considered separately from those collecting the refuse in that district, provide a good illustration of the unit costs of disposal, where a maximum use is made of mechanical appliances for reducing manual labor charges. According to a recent article by Kershaw, the Brookvale Road plant during the calendar year 1925 treated 31,633 long tons of refuse. The total operating cost of the plant was $41,164, or $1.30 per long ton, and the income from salvage materials, $17,361, or $0.55 per long ton. The difference, or net cost, was $23,803, or $0.75 per long ton, which is 20 per cent of the unit cost of collection and disposal of the refuse of the city as a whole.

Except for the presence of rather noticeable amounts of dust in the atmosphere of certain rooms of he disposal works, the sanitation of the entire system appears to be virtually perfect. A constant effort is made, by the use of forced ventilation, to reduce this dust hazard to a minimum, and it was stated by officials of the department that in so far as can be determined, it apparently has had no measurably deleterious effect on the health of the plant works. Odors are prevented by an efficient system of washing the flue gases arising from the incinerators. No perceptible ordor was observable in the vicinity of any of the disposal works visited.

The disposal of refuse in Great Britain, though essentially a measure of sanitation, has many economic aspects which are of interest to visiting sanitarians. In this instance, as in that of sewage disposal, a great deal of the substantial progress made toward a solution of the problem in Great Britain has been due in part to the active stimulation given to the local authorities by the Ministry of Health, and in part to the fact that in a majority of British cities these matters are administered wholly by technical specialists in sanitary engineering, whose position and tenure of office are entirely removed from the exigencies of local politics.

Sand-Clay and Earth Roads

Construction and Maintenance Methods in Saskatchewan Described in Paper Presented Before Canadian Good Roads Association

By H. R. MACKENZIE
Chief Field Engineer, Saskatchewan Department of Highways

The type of sand-clay roads being constructed in the province of Saskatchewan is almost exclusively an artificial sand-clay surface on a sand subgrade.

Method of Construction.—Some specifications for this type of road provide for plowng up sand from the sub-grade to mix with the layer of clay that has been spread on the road surface. Such a process was found to be impracticable for the following reasons. Firstly, the difficulty of plowing through any class of soil to a depth sufficient to bring up material from beneath a surface stratum 8 in. to 10 in. in depth. Secondly, in sandy sections quite frequently the only available clay deposits are found in low areas and consequently the clay is wet when excavated, and it quickly bakes hard when placed on the road surface, and if plowed after being spread, you would have an extremely lumpy material which would be difficult, if not impossible, to mix with the sand so as to form a road surface of uniform thickness and density. Thirdly, where comparatively dry clay is available, the plowing and discing of the entire mass of surfacing material, pulverizes the clay, and pulverized clay is just as objectionable to traffic as pure sand, and as the rainfall in some parts of Saskatchewan during mid-summer is usually light, we have had cases where the road surface was almost impassable for long periods, due to the lack of a sufficiently heavy rain fall to enable the sand-clay mixture to become consolidated, and in the meantime a large part of the surfacing material had blown away, and numerous sand pockets had been developed by traffic, making the completion of a satisfactory sand-clay surface a difficult matter.

In view of these difficulties we found it advisable to place a layer of 8 to 10 in. of clay on the sand sub-grade, and to form a sand-clay surface by pulverizing the surface of the clay and adding sand from side borrow pits, which is mixed with the pulverized clay to form a sand-clay mixture. The layer of clay under the sand-clay mixture gives stability to the road surface and prevents the development of sand pockets, and this clay can at a later date be used in the formation of a sand-clay surface to replace worn-out material.

We have approximately 130 miles of sand-clay road under maintenance, and this type of road is giving excellent service. One section is approximately 28 miles in length and it has remained in good condition since its construction six years ago.

It is interesting to note that certain alkali clays are practically impervious to water. We have sections of sand-clay roads in which ruts do not develop during wet periods, and needless to say we are investigating clay deposits carefully before deciding upon the kind of clay to be used in the construction of a sand-clay surface.

Approximately 2,900 cu. yd. of clay is used to construct one mile of sand-clay surface on a sand subgrade. The average cost per mile of a completed sand-clay surface in Sasketchewan is $3,048.

Maintenance of Sand-Clay Roads.—In regard to the maintenance of a sand-clay road on a sand sub-grade, attention must be given to the fact that as the sub-drainage is practically perfect, the road surface dries more rapidly than in the case of an ordinary earth road, and consequently the sand-clay sections should be dragged earlier than the remaining sections of the road.

Earth Roads.—There are approximately 200,000 miles of road allowances in the settled area of the province of Saskatchewan. The main market system which consists of roads giving direct access to shipping points comprises 25,000 miles, and our provincial highway system, which provides for through routes across the province, consists of slightly more than 7,300 miles of road.

The traffic requirements of Saskatchewan at the present time would not justify the expenditure of large sums of money on a comparatively small mileage of roads. What we require is the largest possible mileage of serviceable roads; consequently our improvements to date have been confined to earth and gravel roads.

The department of highways has assumed the responsibility of constructing and maintaining the 7,300 miles of roads on our provincial highway system, and from 1921 to date we have completed the construction of approximately 3,000 miles of well-constructed and thoroughly drained earth roads. We have been raising the standard annually, and we consider that the class of road which we are now constructing will, in the matters of location, grade, alignment, and drainage facilities, be a satisfactory basis for the higher type roads that the future will demand.

Crowns, Slopes and Alignment of Earth Roads.—We are using a 20 ft. road surface with a slight crown of from 4 to 6 in. Since the spring of 1925 we have been using 3 to 1 slopes on all shallow cuts and on all embankments not protected by guard rails. The slopes on all side ditches adjacent to the roadway are also 3 to 1 or over; consequently at all points not protected by guard rail, a motor car or other vehicle may at reasonable speed run into the ditch or down the slope of a fill without danger of upsetting.

In addition to the safety features of 3 to 1 slopes we found that they greatly reduce the damage caused to the slopes by water discharged from the road surface. We also find it much easier to keep the width of the road surface uniform and in good alignment, as 3 to 1 slopes must be built in place, and are consequently consolidated to a much greater degree than 1½ to 1, or natural slopes.

There are a large number of sloughs in Saskatchewan, and the wave wash on embankments across sloughs is a constant source of trouble to our maintenance department. Here again the 3 to 1 slopes practically form a beach and they are much more resistant to wave action than 1½ to 1 slopes, and they are easier to riprap where wave action is formidable. The easy slopes on side ditches prevent the rapid filling up of the ditch by erosion and facilitate the cutting of weeds.

Apart from the adoption of 3 to 1 slopes, the most important recent revision in our specifications is in regard to alignment. We have adopted 16° as the maximum curvature on our provincial highways excepting in particularly difficult locations, and we are purchasing approximately a half acre of land at right-angled intersections to enable us to construct 16° curves to replace the dangerous 90° turns. Sharp turns on our main highways are almost as good a source of spare parts for local car owners as the much despised level railway crossings, and the travelling public has very heartily approved of our efforts towards elimination of these danger points from our highways.

Costs.—Practically all new construction work on our provincial highways is performed by contract, as day labor crews have not been able to satisfactorily compete with the contractors on a unit cost basis. The amount of common excavation on the average mile of road varies from 6,000 to 8,000 cu. yd., and the average cost per mile of completed earth road is approximately $2,500.

The average unit prices of the various classifications on all contract work performed during the period 1921 to 1926 inclusive, and also the average unit prices for the season of 1926 are as follows:

Classification	Unit	Average Cost 1920-1926 Inclusive	Average Cost 1926 Only
Earth excavation	cu. yd.	$ 0.240	$ 0.213
Loose rock excavation	cu. yd.	0.716	0.646
Solid rock excavation	cu. yd.	1.790	1.543
Overhaul	cu. yd. 100 ft.	0.037	0.032
Riprap	cu. yd.	2.201	2.026
Gravel surfacing	cu. yd.	1.085	0.886
Clay surfacing	cu. yd.	0.735	0.574
Guardrail	lin. ft.	0.339	0.341
Grubbing	acre	27.639	23.800
Clearing	acre	66.159	38.646

City Officials Division of A. R. B. A.

A City Officials' Division of the American Road Builders' Association was organized at a meeting held on June 8. The division will be devoted principally to the standardization of city street construction, operation and maintenance methods.

According to the association, millions of dollars can be saved the cities annually by the standardization of these methods. Examples were cited where the use of economical methods have reduced the cost of street construction in many instances by as much as 50 per cent.

The great concentration of traffic in city areas makes a careful and thorough handling of city street affairs a matter of extreme importance.

The new division will operate by means of committees selected from the leading city officials of the country. These committees will study the various problems of city street construction, maintenance and operation and will report their findings at the 1929 Convention and Road Show of the American Road Builders' Association. A special day will be set aside at this great annual convention for the discussion of city street affairs. Approximately 30,000 road builders will attend.

According to the association, approximately $400,000,000 annually is spent on city streets, or a sum equal to more than one-third of the annual construction and maintenance costs of the entire state and rural highway systems of the United States.

The new division will be the sixth to be organized by the association for the purpose of facilitating the economic construction and operation of roads and streets in North and South America.

Seventh Asphalt Paving Conference to Be Held in New Orleans

At a recent meeting of the Board of Directors, held in Chicago, New Orleans was chosen as the city in which the Seventh Annual Paving Conference, a nation-wide convention of asphalt chemists, engineers, producers, distributors, salesmen and contractors, as well as public officials interested in studying the latest methods evolved in constructing street and road pavements, will be held during the week of Dec. 10 this year. New Orleans was selected over a number of other cities which campaigned for the meeting for nearly six months, including Havana, Montreal, Detroit, Atlantic City, Philadelphia, Cleveland, Minneapolis, Chicago and West Baden, Ind.

The American Association of Asphalt Paving Technologists and The Asphalt Association, as well as other organizations, will hold their annual meetings in New Orleans in connection with the conference. It is expected that from 1,000 to 1,500 engineers and federal, state, county and city road officials, as well as asphalt producers, contractors and machinery manufacturers will attend. The program will consist of discussions extending over a period of several days and will relate to the latest, most effective, and most economical methods evolved for the construction of the several types of asphalt paving.

Six national asphalt paving conferences have been held thus far, the first in Atlanta in 1922, the second at Louisville, Ky., in 1923, the third in Denver in 1924, the fourth in Detroit in 1925, the fifth in Washington, D. C., in 1926, and the sixth again in Atlanta in 1927. It is planned to make the New Orleans conference the most informative and largely attended of any that have been held. J. E. Pennybacker, general manager of The Asphalt Association, a national organization with headquarters in New York City, is the secretary of the conference. Reduced railroad rates, as for previous conference, will be in effect on all railroads for the New Orleans meeting.

A. R. B. A. to Take Part in Mexican Road Congress

The American Road Builders' Association has accepted an invitation of the Mexican Federal Highway Commission and Government to participate in the Second National Highway Congress and Exposition to be held in Mexico City on Oct. 3, 4, 5 and 6.

The invitation to the association comes from Mexico after that country has completed its third year of Federal road construction and on the eve of a large highway construction program that will be participated in by all the Mexican states and cities as well as the Federal Government.

Invitations have also been sent to South and Central American countries to take part in this Congress, and it is expected that delegations will be present from all Pan-American countries.

One of the first objectives of the Mexican Government is to complete the highway from the United States border to Mexico City, a distance of about 800 miles.

The object of the American Road Builders' Association in accepting the Mexican invitation is scientific and educational. The associaton will exhibit the latest methods of highway engineering, showing the plans, maps, details and specifications used in the United States. In addition to this, the most recently developed economic construction methods will be shown by movies and photographs. The most up-to-date road building equipment and materials will be exhibited and arrangements have been made for demonstrations of road building machinery.

In accepting the Mexican invitation to take part in the Second National Highway Congress and Exposition, the Road Builders' Association will act through its Pan-American Division, which was recently organized, and will make available to the road builders of Mexico and other Pan-American countries the latest and most economic practices in road building as now being used in the United States.

Special freight rates have been obtained for shipping road building machinery to Mexico, and reduced passenger fares will be in force. Several special trains to Mexico City will be run for the benefit of those wishing to visit the country at that time.

Committee Approves Vitrified Paving Brick Sizes

The five recognized varieties and sizes of vitrified paving brick have again been approved as standard for the coming year. This action was taken by the U. S. Department of Commerce "Committee on Simplification of Variety and Standards for Paving Brick" at its meeting in Washington, D. C., March 22. The five recognized varieties are:

Plain Wire-Cut Brick	2½ in. x 4	in. x 8½ in.
Plain Wire-Cut Brick	3 in. x 4	in. x 8½ in.
Plain Wire-Cut Brick	3½ in. x 4	in. x 8½ in.
Repressed Lug Brick	4 in. x 3½	in. x 8½ in.
Wire-Cut Lug Brick	4 in. x 3½	in. x 8½ in.

George F. Schleisinger, state director of highways, Ohio, and representing the American Association of State Highway Officials, was elected chairman of the committee to fill the vacancy caused by the resignation of E. J. Mehren, vice-president McGraw-Hill Corporation, who has filled that office since its inception in 1921. Other members of the committee are: P. H. Bates, Bureau of Standards, Washington, D. C.; E. L. Beller, National Paving Brick Manufacturers Association, Chicago; R. Keith Compton, American Society of Civil Engineers, Richmond, Va.; George F. Fisk, American Society for Muncipal Improvements, Buffalo, N. Y.; R. M. Hudson, Division of Simplified Practice, Department of Commerce; T. R. Lawson, American Society for Testing Materials, Troy, N. Y.; E. W. McCullough, Chamber of Commerce of the U. S. A.; O. W. Renkert, American Ceramic Society, Canton, Ohio; P. St. J. Wilson, U. S. Bureau of Public Roads, Washington.

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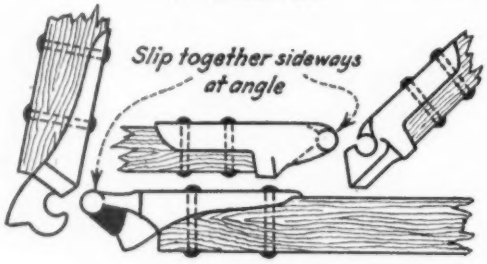

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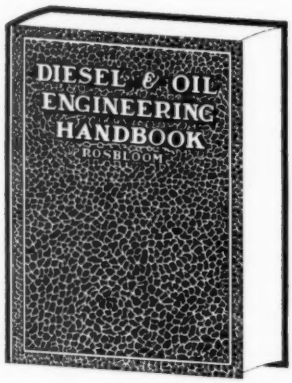

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